Santa Clara Valley Water District

Permanente Creek Flood Protection Project

BASIS OF HYDRAULIC DESIGN

Prepared for
Hatch Mott MacDonald

November 2011

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The following report has been prepared under the supervision of the undersigned, who certifies that he is a Registered Civil Engineer in the State of California.

Prepared by:

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BASIS OF HYDRAULIC DESIGN
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INTRODUCTION

This basis of hydraulic design has been prepared for Hatch Mott MacDonald (HMMD) to document hydraulic analyses of Permanente Creek and the Permanente Diversion Channel for the Permanente Creek Flood Protection Project, which is being designed under the auspices of the Santa Clara Valley Water District (District). The Permanente Creek Flood Protection Project is part of the voter approved Clean Safe Creeks Program. The objectives for the project are to provide flood protection to homes downstream of El Camino Real, prevent the flooding of Middlefield Road and Central Expressway, develop an asset protection plan for the deteriorating concrete channels built previously, and develop long term maintenance guidelines for the creek, while minimizing the cost of maintenance.

This report describes the hydraulic performance of Permanente Creek between San Francisco Bay and Rancho San Antonio County Park in Los Altos, and the Permanente Diversion Channel to Stevens Creek in Mountain View. The development of preliminary designs for three new storage facilities at Rancho San Antonio County Park, Cuesta Park Annex, and McKelvey Park including inlet and outlet structures, emergency spills, and basin drainage facilities at each location are described. Hydraulic and engineering analyses for Permanente Creek and the Permanente Diversion Channel described herein are based upon the District's 60% Plan Set dated February 10, 2010 as subsequently modified by the District and described in their hydraulic models.

Project Objectives

The basic objective of the Permanente Creek Flood Protection Project is to protect properties along Permanente Creek downstream of El Camino Real against the base flood in a manner consistent with the policies and design criteria of the District and the Federal Emergency Management Agency (FEMA) as administered through the National Flood Insurance Program (NFIP). This basic objective will be accomplished in part through the use of off-stream flood detention facilities at Rancho San Antonio County Park near Los Altos; and Cuesta Park Annex and McKelvey Park in Mountain View.

District and NFIP regulations define the "base flood" as a flood having a one percent chance of being equaled or exceeded in any given year. Often this is referred to as a "one-percent" or "100-year" flood. This should not be confused with a flood that *will* occur once every one hundred years, but one that might occur once every one hundred years *on the average* over a very long period of time.

Over the life of a 30-year mortgage, there is a 26 percent chance of experiencing a flood equal or greater in magnitude than the base flood as demonstrated by Table 1. These probability concepts may have an impact on construction staging, especially the initiation and completion of work within particular reaches, and the timing of floodwall construction.

Table 1
Relative Risk of Various Flood Events

	5-year	10-year	100-year
Annual risk of event	20%	10%	1%
Risk of at least one event in 5 years	67%	41%	5%
Risk of at least one event in 10 years	89%	65%	10%
Risk of at least one event in 30 years	99.88%	96%	26%
Risk of at least one event in 50 years	99.998%	99%	39%
Risk of at least one event in 100 years	really high	99.997%	63%

Although the primary project objective is to provide a level of flood protection along Permanente Creek meeting District and FEMA standards, additional project objectives include:

- balancing flood reduction with environmental restoration, community beautification, and recreation opportunities.
- providing efficient and cost-effective flood control maintenance to ensure that the District is able to maintain design system performance in the future; and
- engendering community support for the proposed project.

Scope of Hydrologic and Hydraulic Design

District-supplied hydraulic models for Permanente Creek and the Permanente Diversion Channel for both existing and design conditions have been updated based on the latest available version of HEC-RAS (version 4.1.0), record drawing reviews, and field reconnaissance. Steady-state backwater analyses have been completed to evaluate existing and design capacities within the project reaches. District-supplied hydrologic models have been used to route flood hydrographs through proposed flood protection facilities, thus enabling a comparison of flow to available capacities within project reaches. Unsteady state HEC-RAS models have also been constructed to model the performance of proposed flood storage basins and their accompanying hydraulic structures.

District project reaches are Permanente Creek from San Francisco Bay to the Rancho San Antonio County Park; Hale Creek from the Hale-Permanente Confluence to Foothill Expressway; and the Permanente Diversion Channel to Stevens Creek. Each reach is depicted in Figure 1. Permanente Creek Reach P1 extends from San Francisco Bay upstream to the Boat Pond Bridge at Project Station 62+76; Reach P2 encompasses the improvement project from Boat Pond Bridge upstream to US Highway 101 at Project Station 131+79; Reach P3 extends from Highway 101 upstream to Villa Street at Project Station 200+32; Reach P4 extends from Villa Street to upstream of El Camino Real at Project Station 237+20; Reach P5 extends from upstream of El Camino Real to the Hale-Permanente Confluence near Raymundo Avenue at Project Station 262+91; Reach 6 extends from the confluence upstream to Cuesta Drive at Project Station 302+86; Reach P7 extends from Cuesta Drive upstream to the Permanente Diversion structure at Station 345+24; and Reach P8 extends from the Permanente Diversion structure upstream to Foothill Expressway at Station 395+23. In addition to the original Reaches P1 through P8 as defined by the District, a ninth reach, Reach P9, which extends from Foothill Expressway upstream to Rancho San Antonio County Park at Station 506+32 was defined in order to model the proposed facility at the park. The Permanente Diversion Channel is known as Reach PD. Hale Creek consists of two reaches: Reach H1 from the confluence to Cuesta Drive at Station 4356.95 and H2 from Cuesta Drive to Station 9481.62 at Foothill Expressway.

Sources of Data Used in Analyses

The primary sources of geometric data used in this evaluation have been obtained from the District. These data include field surveys made by District personnel. Some of the data include geometry based on previous hydraulic studies that were submitted to FEMA for floodplain mapping purposes, namely, for reaches P8 and P9. Design discharge hydrographs are as provided directly by the District. Available record drawings have been used to verify some of the data as appropriate.

Basis of Design Outline

After briefly summarizing the facilities outlined in the District's 2008 Permanente Creek Flood Protection Project Planning Study Report, recent project refinements are discussed and routed design discharges are presented for a comparison to updated channel capacities. Each storage basin's preliminary design is described with an evaluation of its effectiveness in attenuating the one-percent flood hydrograph. Alternative post-project one-percent floodplains are compared to the existing conditions floodplain and project objectives. Finally, post-project system performance is examined to demonstrate that the project does not induce significant flooding when runoff exceeds the design discharge.

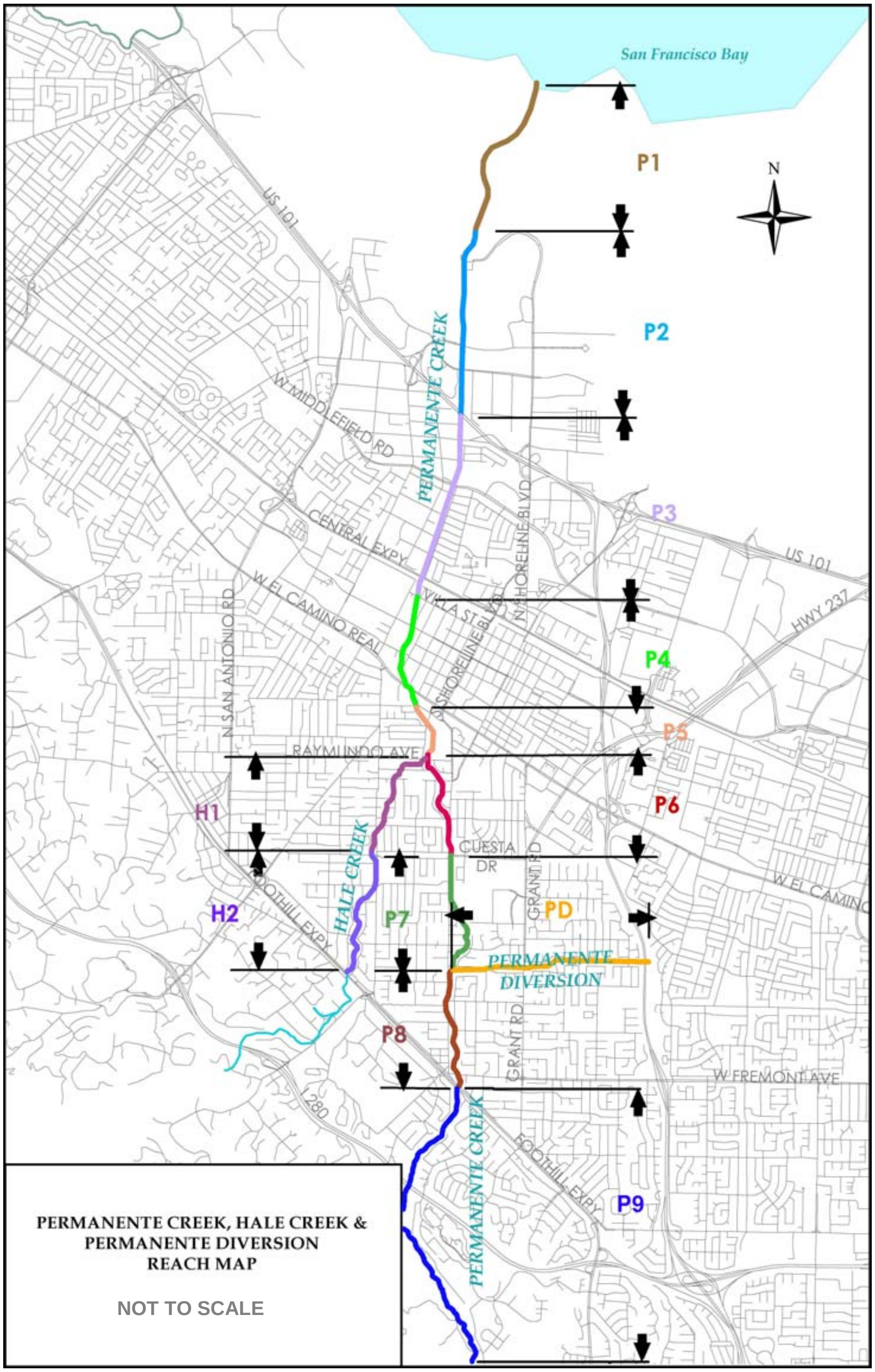


Figure 1: Permanente Creek Reach Designations

PRELIMINARY SYSTEM DESIGN

In July 2008 the District completed the "Permanente Creek Flood Protection Project Planning Study". This report, which is a foundational basis of design for the flood projection project, presents a variety of alternatives that would satisfy the project objectives described previously. After assessing the alternatives for feasibility and environmental impact, and after coordinating project alternatives with interested stakeholders, the District recommended an off-stream storage alternative as best meeting the interests of the public and the District Board of Directors' "Ends Policies".

The recommended alternative consists of off-stream flood detention facilities at Rancho San Antonio County Park, Blach Intermediate School, Cuesta Park Annex and McKelvey Park, in addition to a bypass channel along Hale Creek, channel widening along selected reaches of Permanente Creek and Hale Creek, floodwalls atop levees along Permanente Creek north of U.S. Highway 101, and a new diversion structure at the Permanente Diversion. Figure 2 shows project elements originally outlined in the District Planning Study, as subsequently refined by the District and the Project team. For example, the Rancho San Antonio detention facility has been divided into two separate basins based on detailed topography not available to the District; and the Blach Intermediate School basin has been eliminated.

This Basis of Hydraulic Design addresses the proposed off-stream detention facilities and how they will perform in conjunction with other project improvements, which are to be designed under separate contract.

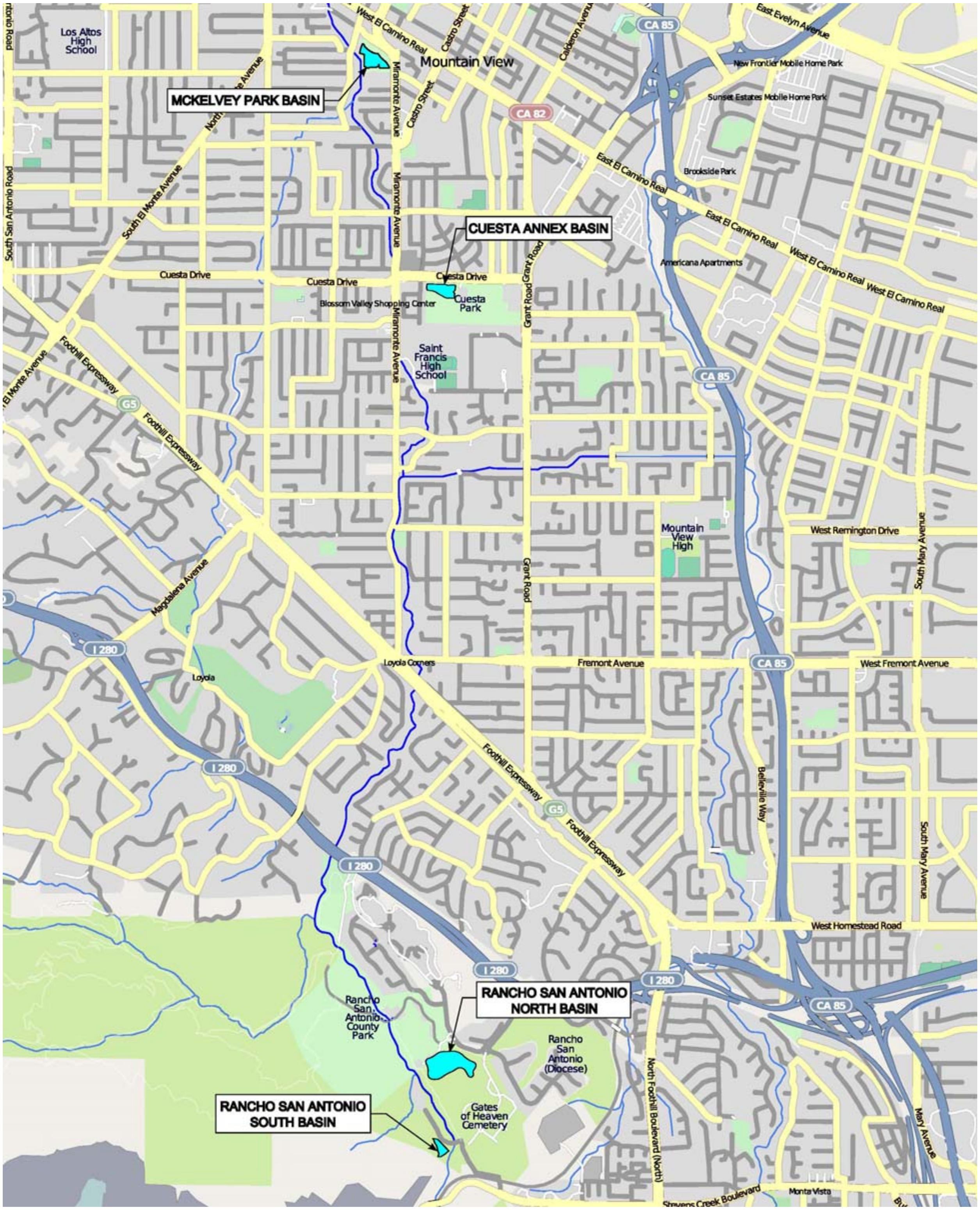


Figure 2: Project Elements

DESIGN DEVELOPMENT

The Permanente Creek Flood Protection Project provides protection from the one-percent flood by storing flood flows that exceed the conveyance capacities of Permanente Creek and the Permanente Diversion. This Basis of Hydraulic Design establishes those capacities and designs storage facilities that meet the project's hydraulic performance objective. In essence this task compares estimated flood flows in Permanente Creek and the Permanente Creek Diversion after proposed storage basins are constructed, to estimated channel capacities with District-planned system improvements, based on the engineering documents referenced above.

Design Discharges

Project design base flood discharges summarized in Table 2 are those provided by the District in October 2011. It should be emphasized that the discharges provided in Table 2 do not necessarily reflect channel capacity limitations. The District's flow estimates assume that all water would be contained or return immediately to the creek. Limited channel capacity also means that ten-year and 100-year discharges are similar in the most downstream reaches.

Table 2
Permanente Creek Design Discharges

Location	Drainage Area (mi^2)	Peak Discharge (cfs)	
		10-year Q10%	100-year Q1%
West Branch Permanente Creek	3.51	452	908
Upper Permanente Creek u/s West Branch Permanente Creek(P1A)	1.17	256	459
Lower Permanente Creek u/s West Branch Permanente Creek	2.48	577	957
Permanente Creek u/s West Branch Permanente Creek(P1)	3.65	757	1,350
Permanente Creek d/s West Branch Permanente Creek	7.16	1,186	2,232
Permanente Creek @ Diversion(P2)	8.05	1,303	2,417
Permanente Diversion to Stevens Creek	8.05	1,303	2,417
Permanente Creek u/s Hale Creek	9.23	108	110
Hale Creek @ Fremont S.C. #33(P3)	2.70	408	744
Hale Creek u/s Permanente Creek	4.39	627	1,075
Permanente Creek d/s Hale Creek(P4)	13.62	811	1,184
Permanente Creek @ Central Expy(P5)	15.40	761	764
Permanente Creek @ US 101(P6)	16.17	830	835

Hydraulic Analyses

Design-level hydraulic models of Permanente Creek are based on District-supplied existing condition and design models. HEC-RAS has been used to create steady state and unsteady state backwater analysis models to determine pre-project reach capacities for existing condition floodplain analyses; post-project reach capacities to evaluate project performance; and to provide design parameters used for the hydraulic structures of the off-stream facilities.

Preliminary hydraulic models of Permanente and Hale Creek have been provided by the District and further developed as part of this work. The project is divided into a series of separate hydraulic models for system analysis.

The upstream model consists of the Permanente Diversion Channel and Permanente Creek upstream of the Diversion Channel to a point upstream of Rancho San Antonio County Park (reaches PD, P8 and P9). The downstream model consists of Permanente Creek from San Francisco Bay to the Permanente Diversion Channel, as well as Hale Creek from the Hale-Permanente Confluence to Foothill Expressway (reaches P1-P7 and H1-H2).

Unsteady state HEC-RAS models have also been created to analyze design inflow and outflow hydrographs at each of the proposed detention facilities. Upstream boundary conditions consist of inflow hydrographs derived from HEC-HMS analyses. Downstream boundary conditions are derived from the steady state model boundary conditions.

HYDRAULIC METHODOLOGIES

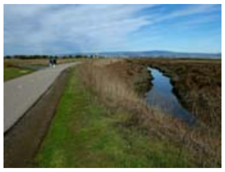
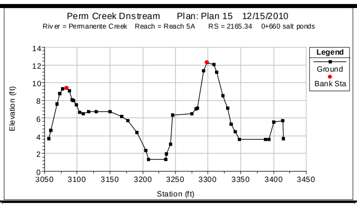
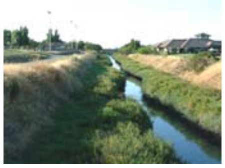
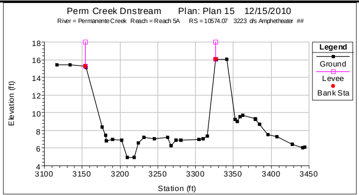
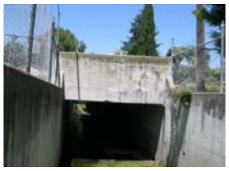
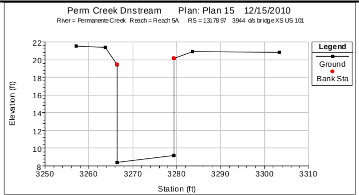
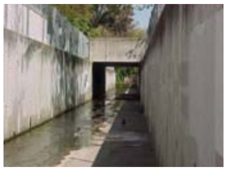
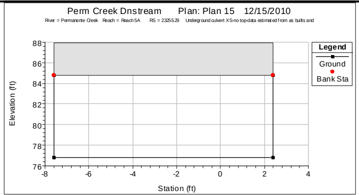
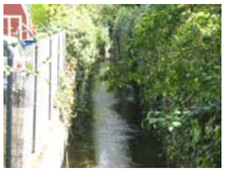
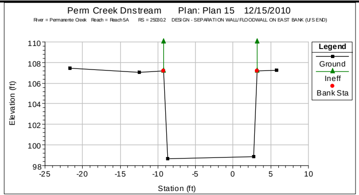
To estimate design reach capacities and to provide design parameters used for the hydraulic structures at the off-stream storage facilities, HEC-RAS models have been prepared to reflect the improvements as described in the 60% design level plans prepared by the District. Relevant hydraulic modeling parameters including channel roughness, bridge modeling, transition losses and boundary conditions are described herein.

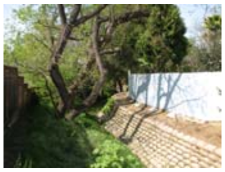
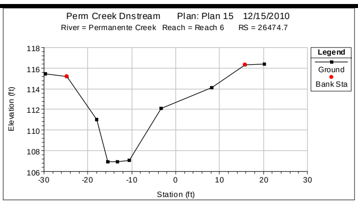
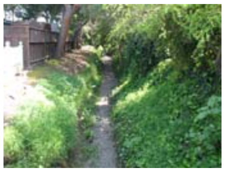
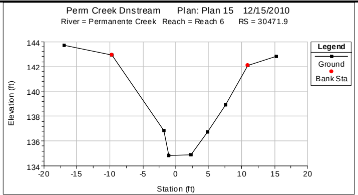
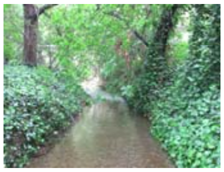
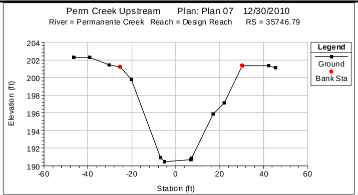
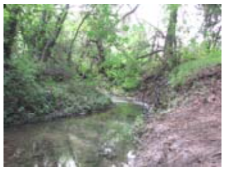
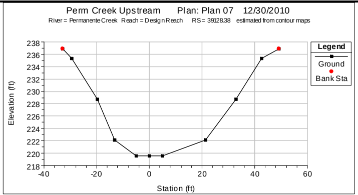
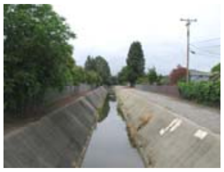
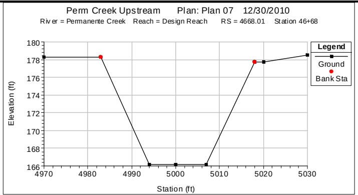
Channel Roughness

In one-dimensional open channel flow analysis, "Manning's n" is used to represent the retarding forces to flow imposed by the channel bed and banks. Roughness elements along the wetted perimeter of Permanente Creek will vary across an individual cross section. For instance, the channel cross section might eventually contain elements of grassed banks, mature riparian vegetation including trees, shrubs or brush, access roads, emergent wetland areas and/or open water, bare earth and concrete walls.

To compute water surface elevations in a channel with variable roughness using a one-dimensional model (i.e. with a mean velocity), it is necessary to estimate an effective (composite) roughness value for each cross section. Table 3 lists estimates of composite roughness used in the channel capacity analysis and illustrates typical cross sections and vegetation representing each project reach of Permanente Creek. Composite roughness coefficients are estimated based on field reconnaissance with environmental conditions projected to winter flood flows, review of literature regarding Manning's "n" values, and previous experience with flood channel and floodplain analyses.

Table 3
Channel Roughness Coefficients Used for Permanente Creek Analysis

Reach	Vegetation	Representative Section	Composite Roughness (Manning's n)
P1			.030
P2			.030
P3			.015
P4			.015
P5			.015

Reach	Vegetation	Representative Section	Composite Roughness (Manning's n)
P6			.035
P7			.040
P8			.045
P9			.050
PD			.015

Transition Losses

An energy loss takes place just upstream and downstream from each structure as flow contracts and expands into and out of a bridge, culvert, or other channel transition. The following contraction and expansion coefficients for channel transitions are used in the hydraulic model (Table 4).

Wherever turbulent conditions create the potential for energy loss, contraction and expansion coefficients are increased. Other transitions include channel bends, resting pools, and maintenance access ramps. For gradual transitions or no transitions between channel sections, the default contraction coefficient (C_i) is 0.1 and the default expansion coefficient (C_o) is 0.3.

Table 4
Contraction and Expansion Coefficients

Transition Type	Contraction	Expansion	Source
Gradual	0.1	0.3	HEC
Warped	0.1	0.2	Chow
Wedge	0.3	0.5	Chow
Square End	0.3	0.75	Chow
Abrupt	0.6	0.80	HEC

Bridge and Culvert Modeling

In addition to the energy required to overcome channel resistance, structures such as bridges and culverts also cause energy losses, which can result in a raised water surface profile. Several methods are available through HEC-RAS to compute energy losses through a bridge. The "energy only" or standard step method handles a bridge section without piers in the same manner as a natural river section, except that the area between the low chord of the bridge (soffit) and the top of road is subtracted from the total cross-sectional area, and the wetted perimeter is increased where water is in contact with the bridge. Increased frictional resistance due to the added wetted perimeter is included in the energy loss through the structure.

When the computed water surface elevation is above the bridge soffit, a "pressure/weir" feature can compute losses through the structure for pressure (orifice) flow, weir flow over the top, or a combination of both. Changes to the water surface profile resulting from the bridge are calculated based on hydraulic formulae that estimate the change in energy and water surface elevation through the bridge. FHWA Chart 8 is used to model the box culvert sections. The energy loss coefficient at the culvert entrance is set to 0.5 reflecting sharp edged culvert entrances with no significant rounding.

Hydraulic Stability

Flood flows can be carried within channels either as subcritical flow or supercritical flow, often referred to as "tranquil" and "rapid" flow, respectively. The flow regime is defined by how fast the water is moving relative to the velocity of the wave that results from a small disturbance in the water surface. Disturbances in subcritical flow move upstream; disturbances in supercritical flow cannot move upstream because such waves must be swept downstream. The Froude number (F_r), which is analogous to the Mach number for gas flow, is defined as the ratio of stream velocity to wave velocity:

$$F_r = \frac{v}{\sqrt{g\,y}}$$

where v = stream velocity (feet per second)
g = gravitational acceleration (feet per second squared)
y = water depth (feet)

A Froude number greater than one signifies supercritical flow (stream velocity greater than wave velocity), while a Froude number less than one indicates subcritical flow. When the Froude number is between 0.8 and 1.2, however, the flow can be unstable, characterized by standing waves and other disturbances that may tend to propagate upstream or downstream depending upon the state of flow. Critical flow is assumed as the upstream boundary condition of each model as well as the downstream boundary condition of the Permanente Diversion.

Tidal Boundary Condition

A coincident one-percent tide analysis has been performed to establish the downstream boundary at San Francisco Bay. The 19-year mean tide cycle for San Francisco Bay represents average tide heights over a specific period known as the tidal epoch, which spans the 19 years it takes for every possible combination of relative positions for the sun, moon and earth to occur. This cycle consists of two high tides (one higher than the other) and two low tides (one lower than the other) each lunar day.

Based on calculations for relative celestial positions, it is possible to predict tides for any day of the year at any time of the day. Astronomic tides, created by the gravitational forces of the moon and sun acting on earth's oceans, are provided in tide prediction calendars. The mean tide cycle is simply the long-term average of astronomic tides. Observed tides, on the other hand, are actual tidal elevations recorded by National Oceanic and Atmospheric Administration (NOAA) gaging stations located throughout coastal areas.

Traditionally, Mean Higher High Water (MHHW) has been used as the backwater condition where riverine (freshwater) runoff meets an estuarine (saltwater) body. However, evidence shows that mean tide elevations are not necessarily an appropriate boundary condition during storm events and tide elevations in San Francisco Bay are elevated (relative to predicted tides) during periods of heavy rainfall. Furthermore, the relationship between coincident tides and maximum annual runoff can be quantified and used in the model, providing for a more statistically correct solution than an arbitrarily selected tide condition.

The El Niño storm of February 2-3, 1998 provided an ideal event for examining potential correlations between runoff events and tide action. While stream runoff as measured by local gages often approached historic recorded levels, observed tides in San Francisco Bay were substantially higher than predicted.

Figure 3 shows predicted and recorded tides in early February 1998 at Redwood City, which has the closest NOAA tide gage to Permanente Creek. Recorded tides during the week of this runoff event were consistently higher (on the order of up to 4 feet) than the astronomic (predicted) tide heights due to storm surge. As a control, observed tide heights are compared to predicted tides six months later at the same station, using the same sets of data. Figure 4 shows tide elevations during early August 1998, when there is very close agreement between the predicted and the actual tides and no rainfall. Both figures present tides on the local Mean Lower Low Water (MLLW) datum.

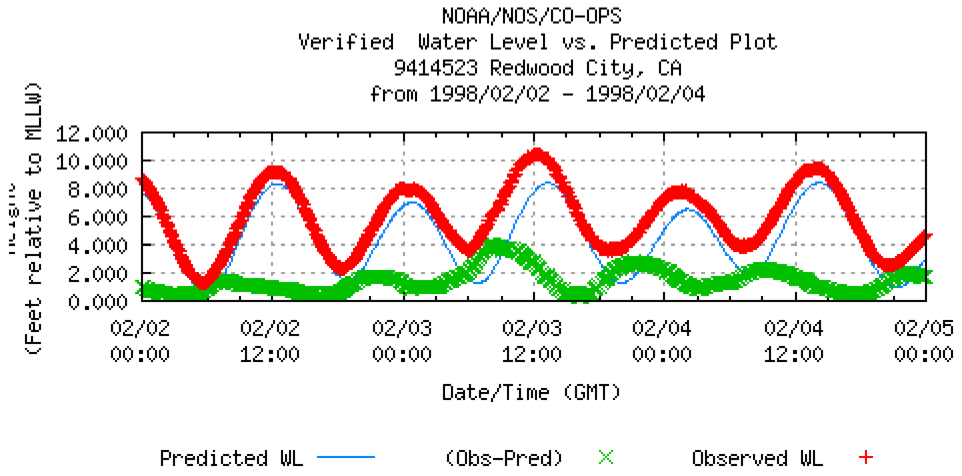


Figure 3: San Francisco Bay Tide During Feb. 2-3, 1998 Storm Event

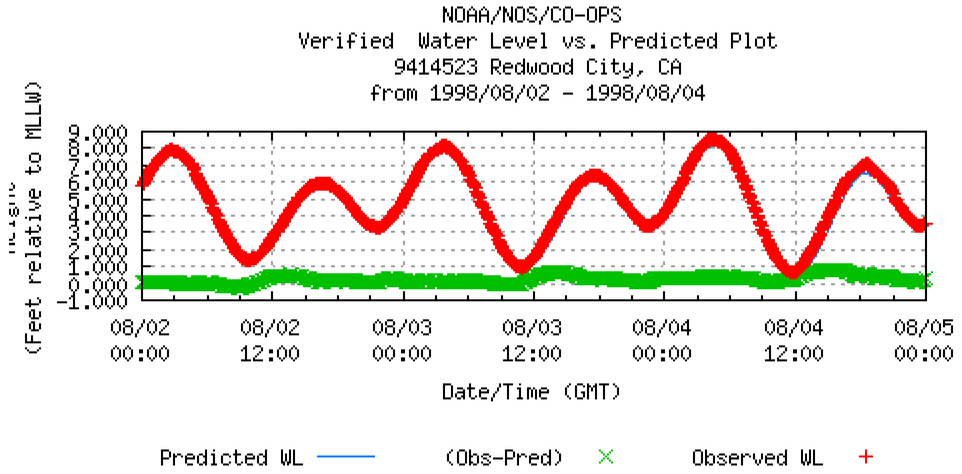


Figure 4: San Francisco Bay Tide During Early August 1998

Historic tide records have been examined to see whether the phenomenon demonstrated in February 1998 at Redwood City occurred elsewhere in the Bay Area and during other heavy runoff events in the past. Results of this investigation presented in Table 5 indicate that during the 1998 runoff event, similar rises in tide elevations (over astronomic) were experienced at other recording tide stations in the Bay.

Table 5
Tides During February 1998 Storm Surge

Location	Maximum Difference Between Predicted and Recorded Tides in feet	
	Higher High	Lower Low
Golden Gate	2.0	2.9
Alameda	2.0	2.7
Redwood City	2.0	2.7
Monterey Harbor	1.7	1.8

The observed phenomenon presented in Table 5 is not strongly dependent upon tide gage location, particularly within San Francisco Bay, and is exhibited during many historic storm events. Data indicate that higher tides as observed during the February 1998 event are not an isolated incident; rather, higher than predicted tides can be expected during storm events that generate significant runoff. Increases in the data set between observed tides over predicted tides range from 0.3 foot to 2.0 feet for the higher high tide, and from 0.9 foot to 3.0 feet for the lower low tide.

From observed historical data, it appears that storm-related forces induce higher tides during rainfall events, and by extension, runoff events. This phenomenon may be due to a number of meteorological or hydrologic factors. NOAA refers to the term “inverse barometer effect”, and defines it as higher tides that are caused by lower barometric pressures associated with winter storm systems. References to “storm surges”, the meteorological effects of low barometric pressures and/or strong southerly winds, are also found in the literature.

The exact nature and cause of this phenomenon, however, are not as important as potential impacts to backwater conditions for Stevens Creek. To model an appropriate San Francisco Bay tidal cycle during the one percent storm event, elevations for each critical point in the tide cycle are adjusted based on the one-percent conditional probability of coincident occurrence with the annual maximum discharge of San Francisquito Creek at Stanford, which represents the closest USGS stream flow gauging location with sufficient length of record for analysis.

This procedure is as described by Dixon (1986), whose hypothesis was that high tide events tend to occur the same day as flood flow events using conditional probability:

$$P(x,y) = P(x|y)\,P(y)$$

where P(x,y) is the probability of occurrence of x and y; P(x|y) is the probability of occurrence of x given y; P(y) is the probability of occurrence of y; x is tide elevation; and y is maximum annual peak discharge. Since we are interested only in annual maximum discharges, P(y) is one and the probability of joint occurrence, P(x,y), is equal to the probability of x given y.

Coincident higher high tides are fitted to a probability curve using the median plotting position for every recorded tide extreme at San Francisco (Presidio/Golden Gate) that occurred within 24 hours of the recorded maximum annual discharge. Figure 5 shows the probability distribution on the MLLW datum.

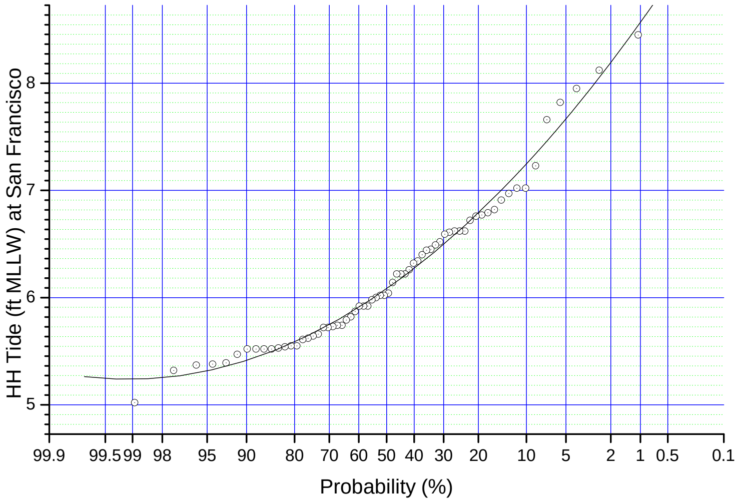


Figure 5: Coincident 1% Tide at San Francisco

The coincident one-percent high tide elevation at San Francisco is 8.63 feet MLLW. A tide elevation at the Golden Gate can be corrected for a different location within San Francisco Bay. The correction at the mouth of Permanente Creek is to add 2.6 feet to high tides. On the MLLW datum, the coincident one-percent tide for Permanente Creek is 11.23 feet. By subtracting 4.1 feet, the tide is converted to 7.13 NGVD (USACE, 1984). Using the Vertcon software, 2.68 feet are added at this location to produce a coincident one-percent high tide at the mouth of Permanente Creek of ***9.8 feet NAVD***.

The timing of coincident tide elevations with peak rainfall/runoff is also a random process. Since there are not sufficient data to statistically analyze the impact of tide timing, it is assumed that peak riverine discharge occurs during the coincident high tide. During the February 1998 event, this turned out to be the case.

System Capacities

Analytical results for estimated channel capacities are summarized by Table 6, which also compares design flows (assuming capacity limitations and spill) to estimates of channel capacity. The analysis of existing channel capacities is based on a completely blocked pipe at the Permanente Diversion (per District direction) that causes all flow in Permanente Creek at that location to be diverted to Stevens Creek.

Floodplain analysis described subsequently shows that spill from Permanente Creek at Foothill Expressway returns to Hale Creek at Foothill Expressway; and spill from Permanente Creek at Portland Avenue returns to the Permanente Diversion Channel opposite Blach School.

Table 6
Existing Channel Capacities and Flows

Creek/Reach/Location	Existing 1% Flow (cfs)	Existing Channel Capacity (cfs)	Channel Spill (cfs)	Location of Spill
PERMANENTE CREEK				
Rancho San Antonio to Foothill Expressway	2,232	2,300	0	
Foothill Expressway to Portland Avenue	2,417	1,500	917	Portland Avenue[1]
Portland Avenue to Diversion	2,417	2,100	0	
Diversion to Cuesta Drive	70	500	0	
Cuesta Drive to Marilyn Drive	70	200	0	
Marilyn Drive to Hale Creek	110	500	0	
Hale Creek to McKelvey Park	1,184	600	584	D/S Hale Creek Confluence
McKelvey Park to El Camino Real	600	800	0	
El Camino Real to Central Expressway	600	1,000	0	
Central Expressway to Bayshore Freeway	764	1,165	0	
Bayshore Freeway to San Francisco Bay	835	1,250	0	
PERMANENTE DIVERSION				
Permanente Creek to Stevens Creek	2,417	1,600	817	

[1]The City of Los Altos plans to upgrade the Portland Ave. Bridge, which is assumed for analysis.

Existing channel capacities are used to compute spill hydrographs for existing conditions floodplain routing. Design channel capacities inform the design of diversion, storage and return flow facilities and are used to evaluate post-project hydraulic performance to ascertain whether the basic project objective is met.

ANALYSIS OF EXISTING FLOOD CONDITIONS

The District has used a two-dimensional unsteady-state numeric model to develop spill hydrographs at locations where the estimated one-percent flow exceeds channel capacity under existing conditions. The District routed spill hydrographs through channel overbank areas to produce a mapped floodplain, to which post-project performance under various scenarios can be compared. A schematic floodplain from the 2008 Planning Study is reproduced as Figure 6. A more detailed map of one-percent flood depths north of Permanente Diversion Channel, prepared by the District using two-dimensional analysis, is shown as Figure 7.

Average base flood depths under existing conditions range from less than one foot to four feet in low lying areas near El Camino Hospital. The predominant depth of flooding is one-half to one foot. The peak spill from the Permanente Diversion Channel near Blach Intermediate School is approximately 820 cfs.

Project Benefit

The main project benefit is to eliminate or reduce these flood hazard zones. All else equal, the elimination or reduction of flood hazards, particularly those presently shown on the effective Flood Insurance Rate Map, enhance property values.

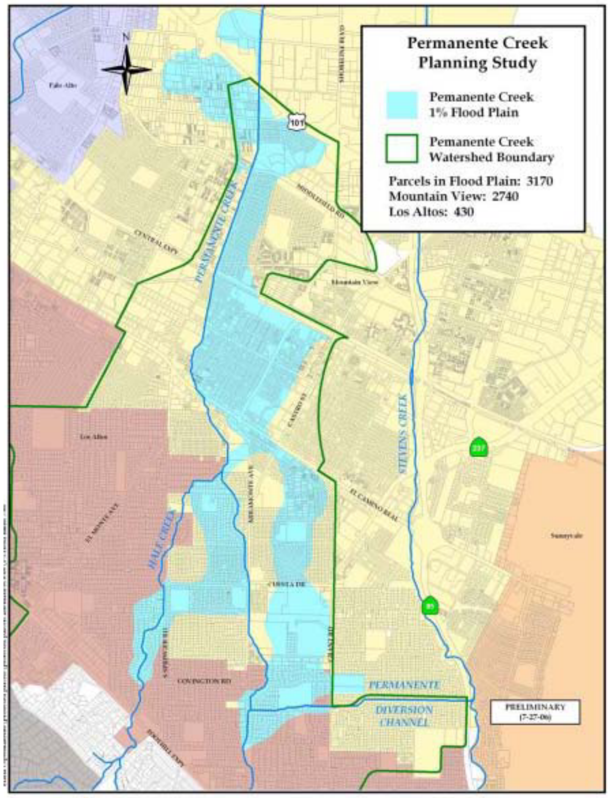


Figure 6: Existing Permanente Creek Floodplain (SCVWD)

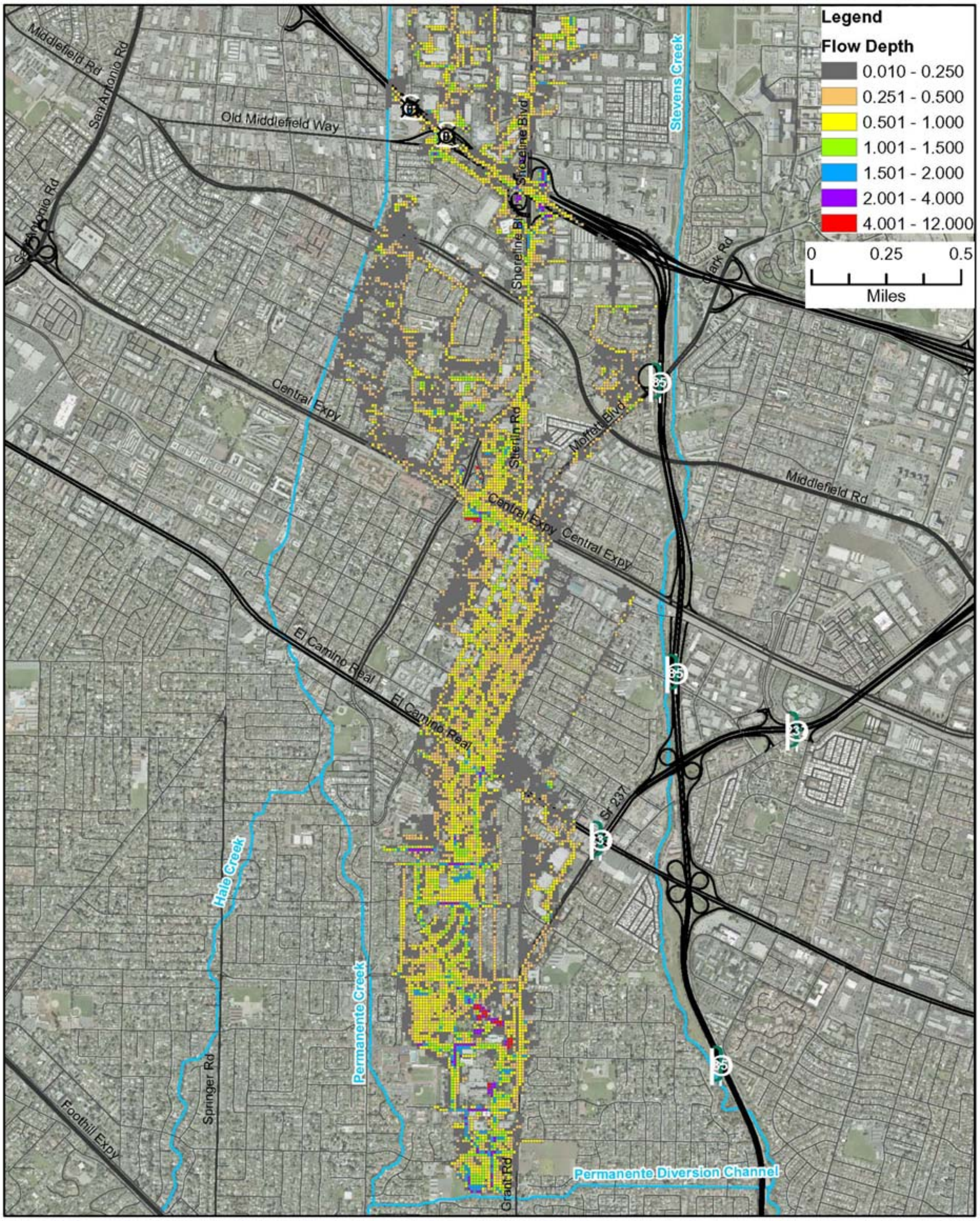


Figure 7: Existing 1% Flooding Depth North of Permanente Diversion Channel (SCVWD)

FLOOD PROTECTION FACILITIES

Detailed inflow, storage-elevation and outflow relationships that govern the hydraulic behavior of each diversion or storage facility are defined and flow hydrographs for the base flood event are routed through each basin using separate HEC-RAS unsteady-state models. Hydraulic structures including lateral weirs, inlet and outlet control structures, overflow weirs, and culverts have been designed and optimized using the unsteady models. Outflow hydrographs from each storage basin model are then input into HEC-HMS to accurately route flow down the Permanente Creek Flood Protection project reaches. Design criteria for each of the storage basins are described in this section. Conceptual designs for each facility are located in the Appendices.

Storage-elevation curves have been provided by Hatch Mott MacDonald, and are based on preliminary design concepts that evaluate Project requirements including recreational use. Much iteration has gone into designing the inflow, outflow and storage facilities; but for brevity, only the final alternative 30% level designs are documented herein. It is presumed that additional hydraulic modeling will be performed as the project team completes the design development phase of the Project.

Overview

Several alternatives remain viable that meet the District's basic objective; that is, the removal of one-percent flooding north of El Camino Real. The design approach is to maximize upstream system storage at Rancho San Antonio Park, given the topographic constraints, a desire to avoid Department of Safety of Dams jurisdiction, and the recreational and aesthetic needs of Santa Clara County Parks.

The effectiveness of this upstream detention to reduce one-percent flows to below the hydraulic capacity of Permanente Creek between Rancho San Antonio Park and the Permanente Diversion is examined first. Then additional diversion and storage alternatives are formulated based on Permanente Creek channel capacity limitations between Cuesta Drive and the confluence with Hale Creek. Hydraulically viable alternatives include the following:

1. Limit flow downstream of the Permanente Diversion to 100 cfs ± as previously planned by the District. This results in a spill from the Permanente Diversion Channel at Blach School and a residual floodplain. To prevent the floodplain from extending north of El Camino Real, shallow floodwaters could be intercepted and returned to Permanente Creek at a) Cuesta Drive; b) Barbara Avenue; c) Hans Avenue or d) El Camino Real.

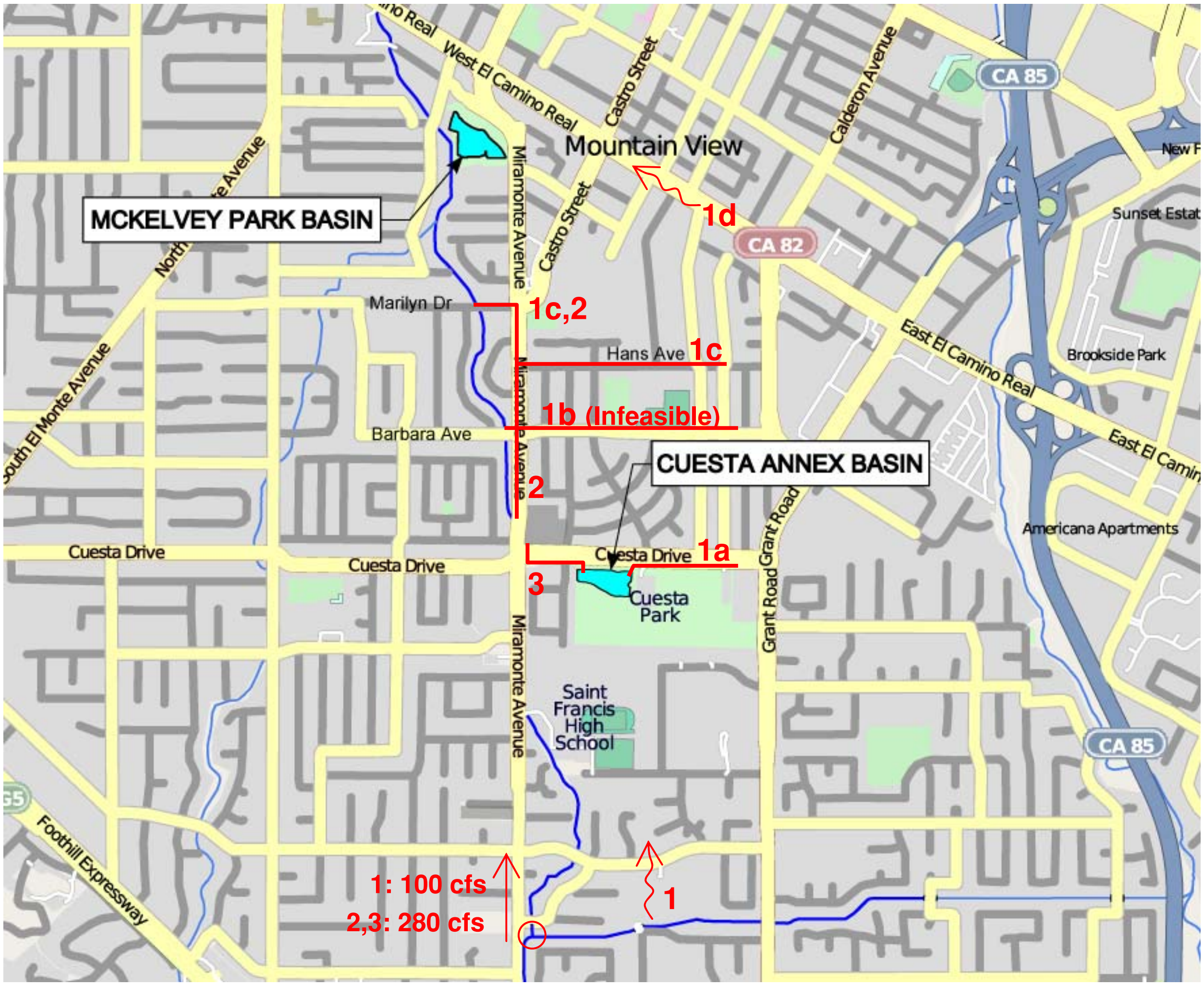
2. Allow more flow downstream of the Permanente Diversion (280 cfs ±) to vastly reduce or eliminate spill from the Diversion Channel at Blach Intermediate School. Install a bypass in Miramonte Avenue to divert flow around the limited channel capacity reach of Permanente Creek between Cuesta Drive and Marilyn Drive. This is the most direct hydraulic solution and eliminates the need for a storage basin at the Cuesta Park Annex. However, Miramonte Avenue is already congested with underground utilities and this may not be a technically feasible or cost-effective solution. Also, without flood attenuation at Cuesta Park, additional storage is required at the McKelvey Park basin.

3. Divert excess flow from the creek at Cuesta Drive. In this alternative about 280 cfs would also be diverted into the creek from the Permanente Diversion Channel, leaving a negligible one-percent spill from the Permanente Diversion at Blach Intermediate School and no significant residual floodplain. Flow at Cuesta Drive that exceeds downstream capacity within Permanente Creek would be diverted over an underground weir structure built within existing District right-of-way on the east side of Miramonte Avenue and into a storage basin built within the Cuesta Park Annex. This weir would limit "normally occurring" basin water to that generated from within the basin itself. A permeable bottom should allow this water to normally infiltrate and reduce potential problems with vector control. The Cuesta Park Annex basin would drain by gravity through a flapgated opening within the diversion structure at Cuesta Drive and Miramonte Avenue.

Viable flow diversion alternatives are shown on Figure 8 .

Finally, the McKelvey Park storage basin is analyzed based on the alternative design hydrographs at the confluence of Permanente Creek with Hale Creek. It is noted that given the relative discharges at this location, the hydrograph from Hale Creek dominates the performance of the McKelvey Park basin; however, basin design is influenced by the alternative selected for Permanente Creek diversion.

It is desirable that infrequently stored floodwaters in McKelvey Park return to Permanente Creek through gravity drainage, while acknowledging that some low-flow pumping may be required for proper park drainage and sub-drainage.



1: Divert 100 cfs at Permanente Diversion Structure and allow spill to occur at Blach School
1a: Same as 1, except catch residual floodplain at Cuesta Drive and direct to Cuesta Annex Basin before releasing to Permanente Creek
1b: Same as 1, except catch residual floodplain at Barbara Ave and pipe to Permanente Creek
1c: Same as 1, except catch residual floodplain at Hans Ave and pipe to Pipe to Permanente Creek
1d: Same as 1, except allow residual floodplain to enter storm drain system on El Camino Real
2: Divert 280 cfs at Diversion Structure to prevent spill at Blach School, and bypass Permanente Creek between Cuesta Drive and Marilyn Drive
3: Divert 280 cfs at Permanente Diversion Structure to prevent spill at Blach School, and divert excess flows to Cuesta Annex Basin

Figure 8: Permanente Diversion Alternatives

Rancho San Antonio County Park

To provide enough storage capacity to adequately reduce downstream system flows relative to system capacity, the facility at Rancho San Antonio County Park consists of two separate storage basins. A 91 acre-foot basin, referred to as the North Basin, is located north of The Gate of Heaven Cemetery on the east side of Permanente Creek. A smaller 9 acre-foot basin, referred to as the South Basin, is located to the south of the large basin on the west side of the creek. A creek flow diversion structure will be located at the Gate of Heaven Cemetery maintenance bridge. Appendix A presents more detailed design parameters for the Rancho San Antonio County Park storage basins.

Storage-elevation data for the two basins are presented in Table 7, and the corresponding storage-elevation curves are shown in Figure 9. It should be emphasized that these are separate basins and while the total amount of design storage is 100 acre-feet, they act independently.

Table 7
Storage-Elevation Data for Rancho San Antonio Basins

Rancho San Antonio South		Rancho San Antonio North	
Elevation (ft)	**Storage (ac-ft)**	**Elevation (ft)**	**Storage (ac-ft)**
434	0.0	394	0
435	0.04	395	4.5
436	0.15	396	9.2
437	0.37	397	14.1
438	0.71	398	19.2
439	1.24	399	24.6
440	1.98	400	30.1
441	2.89	401	35.9
442	4.00	402	41.9
443	5.34	403	48.1
444	6.90	404	54.5
445	8.62	405	61.2
		406	68.1
		407	75.3
		408	82.7
		409	91.0

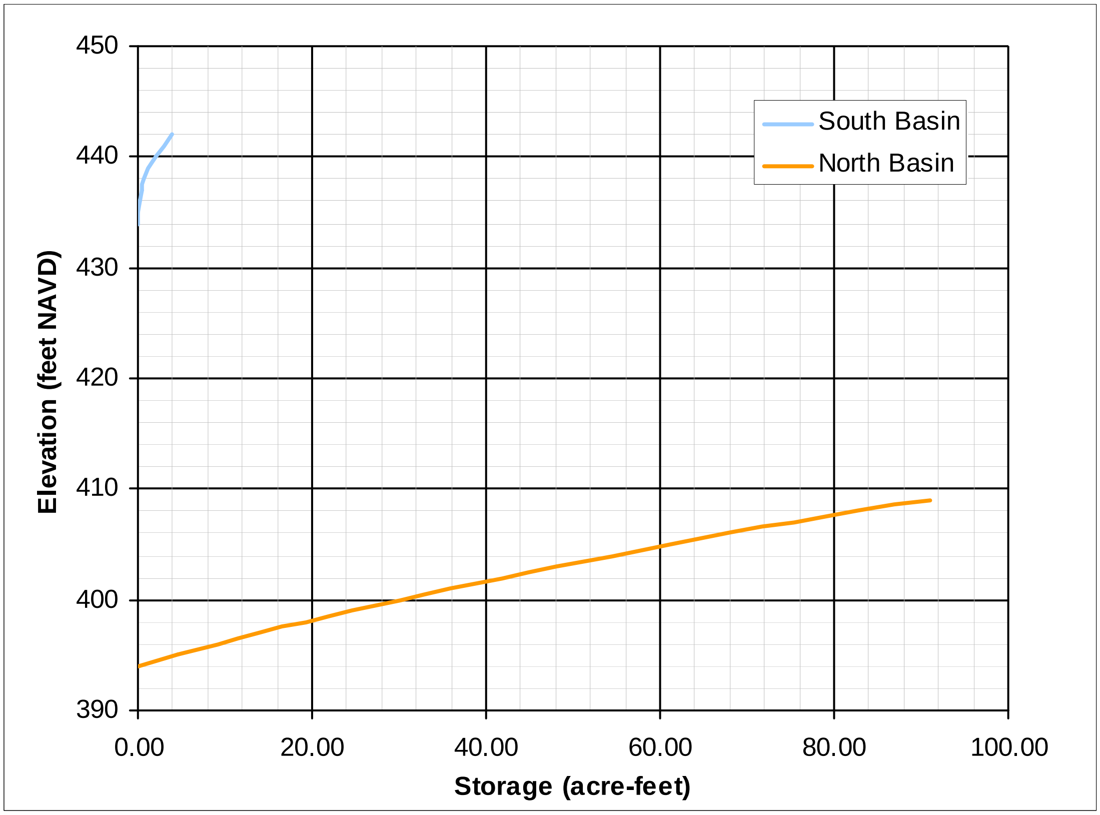


Figure 9: Storage-Elevation Curves for Rancho San Antonio Basins

To force overflows into the two storage basins, a culvert control structure that raises the existing top of the road crossing from an elevation of 437 feet NAVD to an elevation of 445.5 feet NAVD is necessary. This structure could be created using an earthen berm with stable side slopes or with a structural retaining wall. (An earthen berm is shown on the conceptual plans.) The Project team will need to consider cost, construction, function, environmental impact and aesthetics when determining the preferred configuration. A 7-foot by 6-foot box culvert installed within the crossing serves as a hydraulic choke that limits downstream discharge. The approximate length of the culvert assuming an earthen berm is 50 feet. Wing walls that also function as lateral diversion weirs will be installed on both banks of the creek.

(Early design concepts attempted to use simple box intakes at the existing maintenance road crossing to minimize the footprint of new construction. This concept, however, did not provide for the proper split of diverted flow between the two basins and remained hydraulically unstable.)

On the west side of the creek, a 100-foot long wing wall diverts water to the South Basin. Water that overtops the weir at elevation 443.5 feet NAVD is contained in a concrete structure connected to the basin, which has a bottom elevation of 435.0 feet NAVD, via (4) 36-inch diameter pipes. Energy from the water's free fall is dissipated through the pipes, with outlets level to the basin floor. To avoid damage to the exposed basin soils, rock riprap protection is placed at the outlets.

On the east side of the creek, a 95-foot long wing wall at elevation 442.6 feet NAVD88 diverts water into a 12-foot by 8-foot box culvert structure that follows the Coyote Trail to the North Basin. This box culvert, which could be poured in place or constructed from precast concrete sections, has a longitudinal slope of 0.16 percent to avoid excessive flow velocities. Four 8-foot vertical drop structures are proposed to compensate for the remaining 32 vertical feet of grade between the creek diversion structure and the bottom of the North basin, which is at elevation 394.0 feet NAVD. The total length of culvert is approximately 1,500 lineal feet. Conceptual plans show the drop structures constructed by simply overlapping the end of one 12-foot by 8-foot box culvert over another 12-foot by 8-foot box culvert, with matching 10-foot square openings in the respective floor and roof. At the storage basin outfall, the culvert size is increased to 12-foot by 12-foot to prevent highly erosive exit flow velocities, and a recessed security rack is included for public safety.

Both the North and South Basin will need emergency spillways to protect embankments against scour from inflows that exceed the anticipated 100-year inflow. The South Basin's spillway crest will be approximately 50 feet in length and the North Basin's crest will be 750 feet in length. Overflows will flow back into Permanente Creek at each basin. It is anticipated that the landscape architect can create an emergency spillway that does not appear to be an emergency spillway. Some form of scour protection, potentially vegetated geogrid, block, or rock is needed to protect the embankment.

Rancho San Antonio Basin Performance

System performance is evaluated with each of the two previously described basins constructed and functional. Flood flow routing through each basin is modeled using the unsteady function of HEC-RAS. The resulting discharge hydrographs are routed through the stream system using HEC-HMS to evaluate system performance; that is, whether the limiting flow capacity in Permanente Creek or the Permanente Diversion Channel is exceeded at any downstream location. A final post-project design peak 100-year water surface profile is calculated for the entire system using the steady state version of HEC-RAS (Appendix D).

The main Project objective is to eliminate or reduce special flood hazard areas adjacent to Permanente Creek for the base (100-year) flood event. This is accomplished by diverting and temporarily storing flood flows in excess of downstream channel capacity. Hydraulic analysis assumes the City of Los Altos' proposed reconstruction of the Portland Avenue Bridge is complete, and that any channel overflow into the right (east) overbank is contained within the adjacent Heritage Oaks Park.

Table 8 summarizes post-project flow routing along Permanente Creek from Rancho San Antonio Park downstream to the confluence with the Permanente Diversion. Design capacity refers to the post-improvement capacity rather than the existing channel capacity from Table 6. The total amount of stored water in the system is 100 acre-feet, compared to the 93 acre-feet of storage envisioned for Rancho San Antonio Park by the District in their 2008 Planning Study.

Table 8
Flow Routing of from Rancho San Antonio Park to Permanente Diversion

Creek/Reach/Location	Design Capacity (cfs)	Existing 1% Flow (cfs)	Design 1% Discharge (cfs)	Inflow (cfs)	Return Flow (cfs)	Target Storage (ac-ft)
PERMANENTE CREEK						
Upstream of Rancho San Antonio Park	n/a	1,350	1,350	-	-	-
South Rancho San Antonio Park Basin	-	-	-	180	34	8.62
Rancho San Antonio Park	2,300	1,350	711			
North Rancho San Antonio Park Basin	-	-	-	544	43	91.0
Rancho San Antonio to Foothill Expressway	2,300	2,232	1,691	-	-	-
Foothill Expressway to Portland Avenue	1,500[1]	2,417	1,691	-	-	-
Portland Avenue to Diversion	2,100	2,417	1,878	-	-	-

[1] Spill at Portland Avenue contained within Heritage Oaks Park

Flow is diverted at Rancho San Antonio County Park to reduce peak discharges through the capacity-limited reaches. Figure 10 shows the 100-year flow hydrographs at The Gate of Heaven Cemetery diversion structure, which are summarized by Table 9.

Figure 11 shows the 100-year stage hydrographs for the North Basin and South Basin, respectively. It is noted that the North Basin empties within 37 hours of first spill, and the South Basin empties within 6 hours. This performance is well within Santa Clara County's general requirement that detained water be drained within three to five days for vector control.

Figure 12 demonstrates how design hydrographs are routed from above Rancho San Antonio Park to Interstate 280 and downstream to the Permanente Diversion.

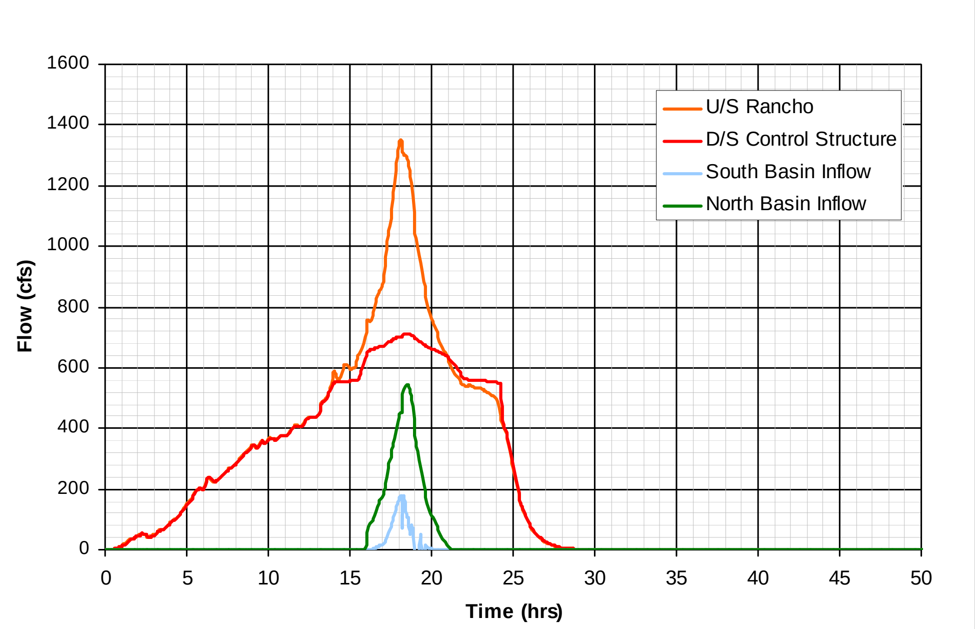


Figure 10: Flow Diversion at Rancho San Antonio County Park

Table 9
Summary of Rancho San Antonio County Park Flood Routing

Routing Parameter	Value
CONTROL STRUCTURE	
Peak Upstream Discharge on Permanente Creek	1,350 cfs
Peak Discharge through Control Structure	711 cfs
Peak Discharge to North Storage Basin	544 cfs
Peak Discharge to South Storage Basin	180 cfs
Peak Recombined Flow Downstream on Permanente Creek	784 cfs
NORTH STORAGE BASIN	
Inflow Volume	99.1 ac-ft
Retained Volume	88.3 ac-ft
Return Volume	99.1 ac-ft
Maximum Elevation in Basin	408.5 ft
Peak Return Flow	43 cfs
Time to Drain	37 hrs
SOUTH STORAGE BASIN	
Inflow Volume	13 ac-ft
Retained Volume	8.0 ac-ft
Return Volume	13.4 ac-ft
Maximum Elevation in Basin	444.6 ft
Peak Return Flow	34 cfs
Time to Drain	6 hrs

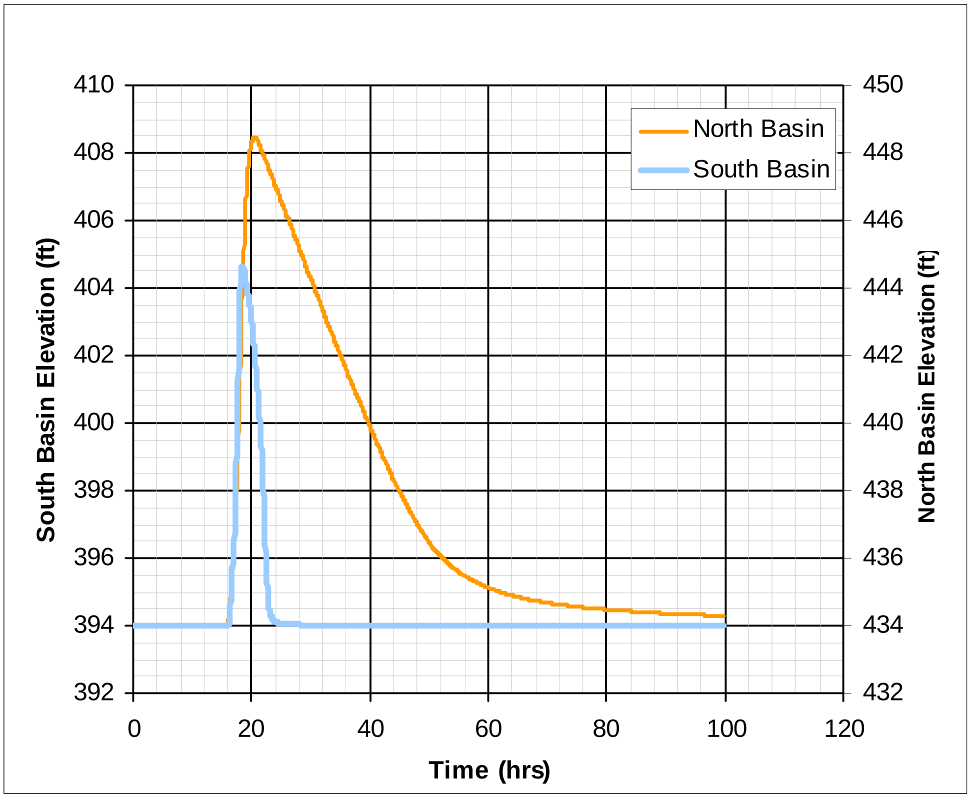


Figure 11: Stage Hydrographs at Rancho San Antonio County Park Basins

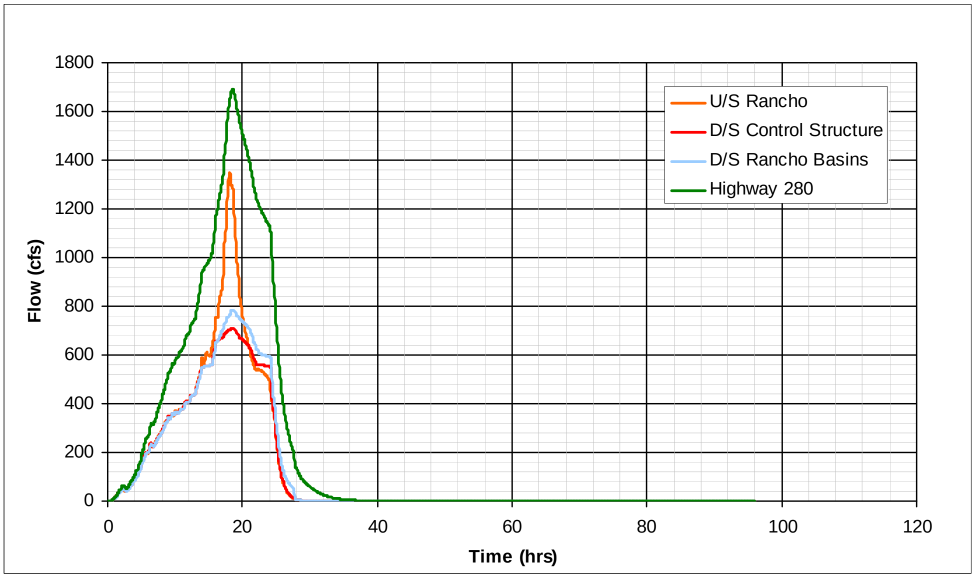


Figure 12: Hydrograph Routing to Permanente Diversion

Permanente Diversion Alternatives

Based on hydrograph routing described previously, the peak discharge at the confluence of Permanente Creek with the Permanente Diversion is 1,878 cfs. Hydraulic analysis using an unsteady state flow model indicates that the capacity of the Diversion channel between Permanente Creek and Stevens Creek is roughly 1,600 cfs. Additional hydraulic analysis concludes that the limiting capacity of Permanente Creek between the Diversion and the confluence with Hale Creek is about 200 cfs, due to bottlenecks located between Cuesta Drive and Marilyn Drive. Furthermore, local runoff enters Permanente Creek within this capacity constricted reach. Thus, regardless of how the discharge is split between Permanente Creek and the Diversion, the total flow is in excess of total system capacity. Therefore flow must be removed from the system in some alternate form of diversion as introduced previously and shown in Figure 8.

Alternative 1: Restrict Flow at Permanente Diversion

For this alternative, which might be considered a relatively minimalist alternative, the Permanente Creek capacity restriction between Cuesta Drive and Marilyn Avenue is respected by limiting the diversion of flow into the natural creek to approximately 100 cfs as shown in Figure 13. This alternative is also most similar to the originally selected District alternative from their 2008 planning study, although the previously proposed Blach Intermediate School storage basin has been eliminated from the project.

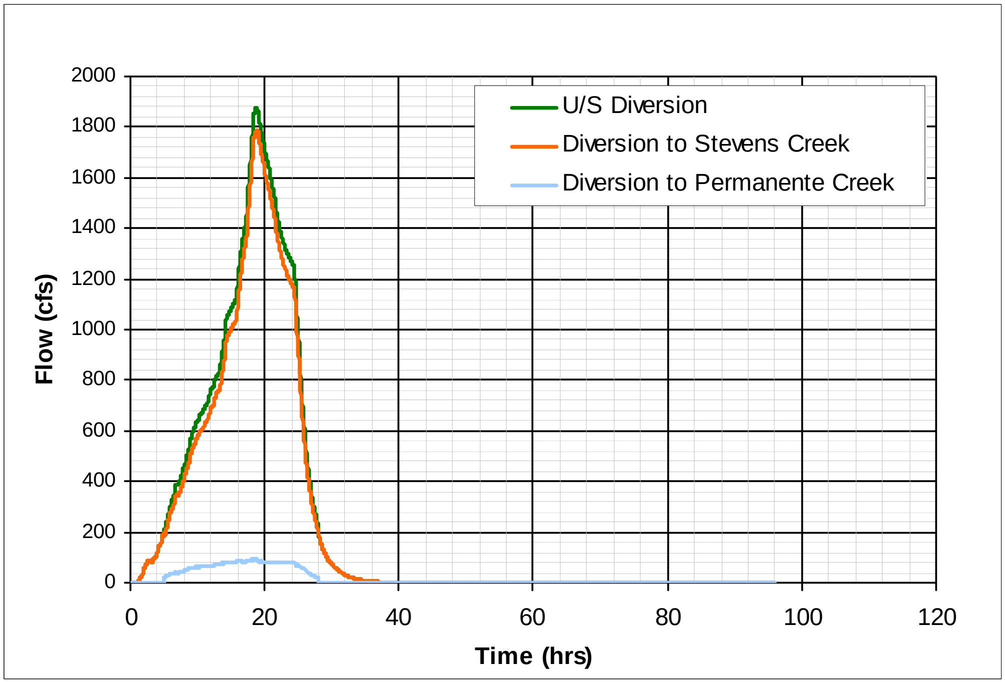


Figure 13: Restricted Flow at Permanente Diversion Channel

With diverted flow limited to 100 cfs, District hydraulic analysis indicates there would be a spill out of the Permanente Diversion of roughly 220 cfs near Blach Intermediate School. Although this spill is less than the existing conditions one-percent spill shown in Figure 7 (approximately 820 cfs), some properties between Blach Intermediate School and Cuesta Drive would remain in a 100-year flood hazard zone after project completion. The project alternative described herein consists of a flood flow cutoff feature that would intercept the potential 100-year floodplain at one of the potential locations described as sub-alternatives below and summarized by Table 10.

Table 10
Residual Floodplain Cutoff Alternative Summary

Cutoff Location	Intercepted Flow (cfs)	Pipe Diameter (inches)	Pipe Length (feet)	Feasible?	Required Storage (ac-ft)
1a. Cuesta Drive	60	54	1,600	No	6.8
1b. Barbara Avenue	32	n/a	2,100	No	n/a
1c. Hans Avenue	14	30	3,300	Yes	n/a
1d. El Camino Real	6	24	4,000	Yes	n/a

Alternative 1a – Intercept Residual Floodplain at Cuesta Drive

The District performed two-dimensional modeling using FLO2D and estimated that the 220 cfs spill would be attenuated through floodplain storage to approximately 60 cfs at Cuesta Drive. If the intercepted flow is returned directly to Permanente Creek at Cuesta Drive, the combined routed peak discharge is about 210 cfs (Figure 14), which would exceed the capacity of the creek channel between Cuesta Drive and Marilyn Avenue. Therefore flood storage within the Cuesta Annex Basin (see Cuesta Park Annex discussion below) would be required for Alternative 1a, to sufficiently reduce the flow in Permanente Creek for safe conveyance to the Hale Creek confluence.

The residual one-percent floodplain for this sub-alternative is shown as Figure 15. The District's two-dimensional model indicates that floodplain storage attenuates the peak discharge to about 60 cfs at Cuesta Drive. To accommodate this flow, the cutoff would consist of a buried 54-inch diameter storm drain pipe aligned along Cuesta Drive from Grant Road to the proposed Cuesta Park Annex storage basin. The length of pipe would be approximately 1,600 lineal feet.

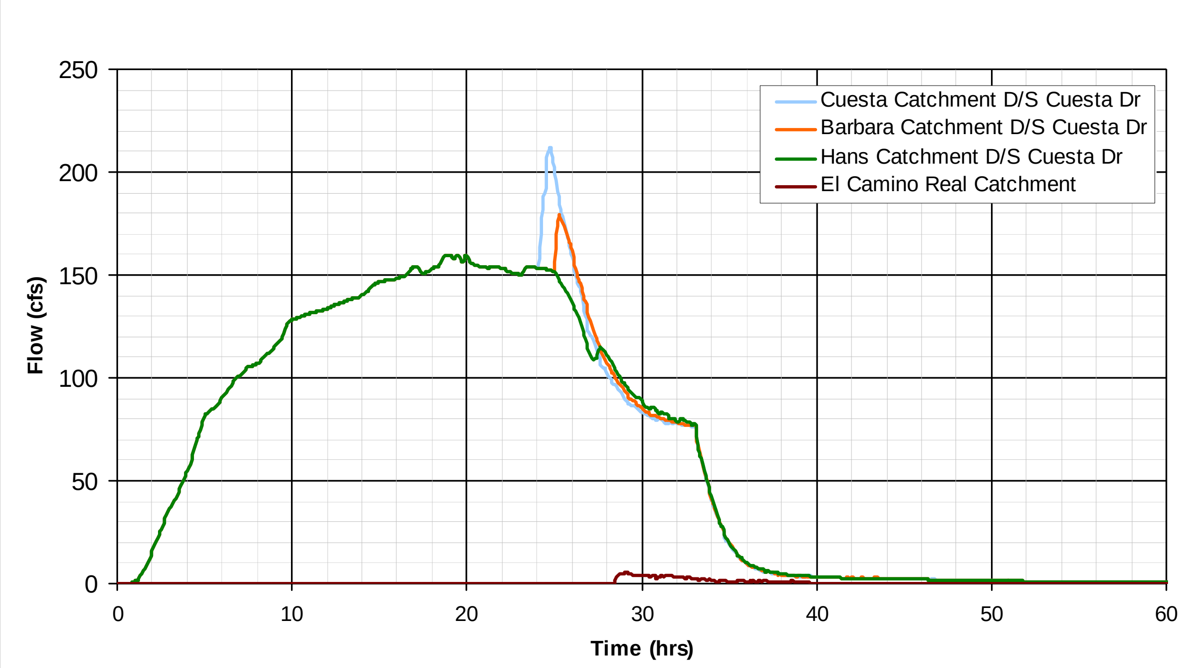


Figure 14: Discharge in Permanente Creek with Floodplain Cutoff Alternatives

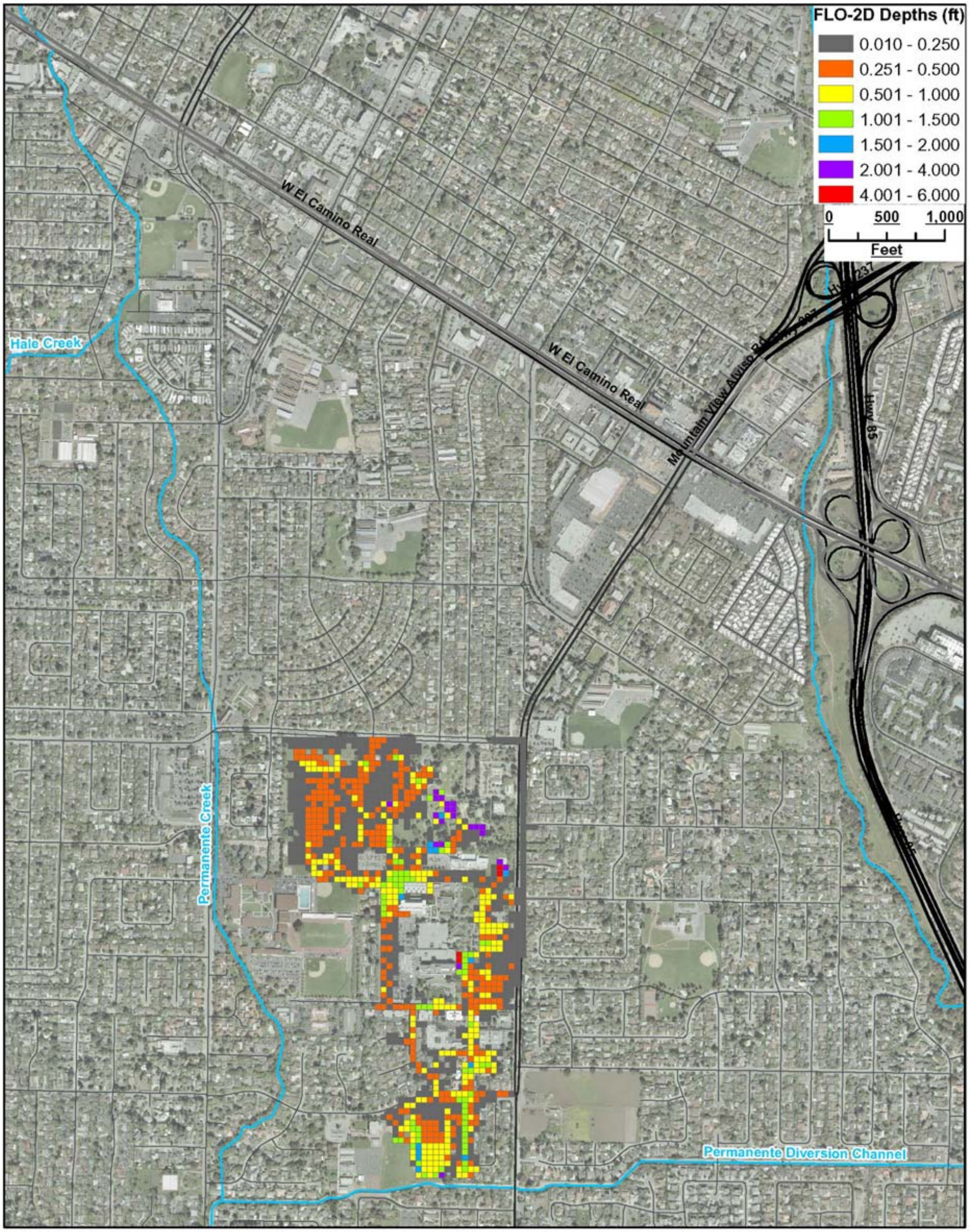


Figure 15: Residual Floodplain with Cuesta Drive Cutoff

Alternative 1b – Intercept Residual Floodplain at Barbara Avenue

The residual one-percent floodplain for this sub-alternative is shown as Figure 16. The District's two-dimensional model indicates that floodplain storage attenuates the peak discharge to about 32 cfs at Barbara Avenue. The requisite cutoff pipe would be approximately 2,100 lineal feet along Barbara Avenue from Bonita Avenue to a new outfall at Permanente Creek. However, this storm drain would be at adverse grade, and the one-percent water surface elevation at the potential outfall location is greater in elevation than natural ground along this alignment. While the combined routed peak discharge of 180 cfs (Figure 14) would not exceed the capacity of the creek channel between Cuesta Drive and Marilyn Drive, due to the adverse grade along the potential interceptor alignment, this sub-alternative is not hydraulically feasible.

Alternative 1c– Intercept Residual Floodplain at Hans Avenue

The residual one-percent floodplain for this sub-alternative is shown as Figure 17. The District's two-dimensional model indicates that floodplain storage attenuates the peak discharge to about 14 cfs at Hans Avenue. To accommodate this flow, the cutoff would consist of a buried 30-inch diameter storm drain pipe aligned along Hans Avenue from Nilda Avenue, along Miramonte Avenue from Hans Avenue to Marilyn Drive, and west on Marilyn Drive to a new outfall at Permanente Creek. The total length of pipe would be approximately 3,300 lineal feet.

If the intercepted flow is returned directly to Permanente Creek at the new Marilyn Drive outfall, the combined routed peak discharge is about 160 cfs (Figure 14), which would not exceed the capacity of the creek channel downstream from Marilyn Drive. Therefore, this sub-alternative is hydraulically feasible and there is sufficient head to deliver water into Permanente Creek when it is at its peak one-percent stage at the potential outfall location.

Alternative 1d– Intercept Residual Floodplain at El Camino Real

The residual one-percent floodplain for this sub-alternative is shown as Figure 18. The District's two-dimensional model indicates that floodplain storage attenuates the peak discharge to about 6 cfs at El Camino Real. To accommodate this flow, the cutoff could consist of a buried 24-inch diameter storm drain pipe aligned within eastbound Caltrans right-of-way from between Ehrhorn Avenue and Bush Street to a new outfall at Permanente Creek. The total length of pipe would be approximately 4,000 lineal feet. Given the small amount of remaining flow, return discharge to the creek would not pose a capacity problem. It also appears that the existing eastbound street section has sufficient capacity to safely intercept this runoff.

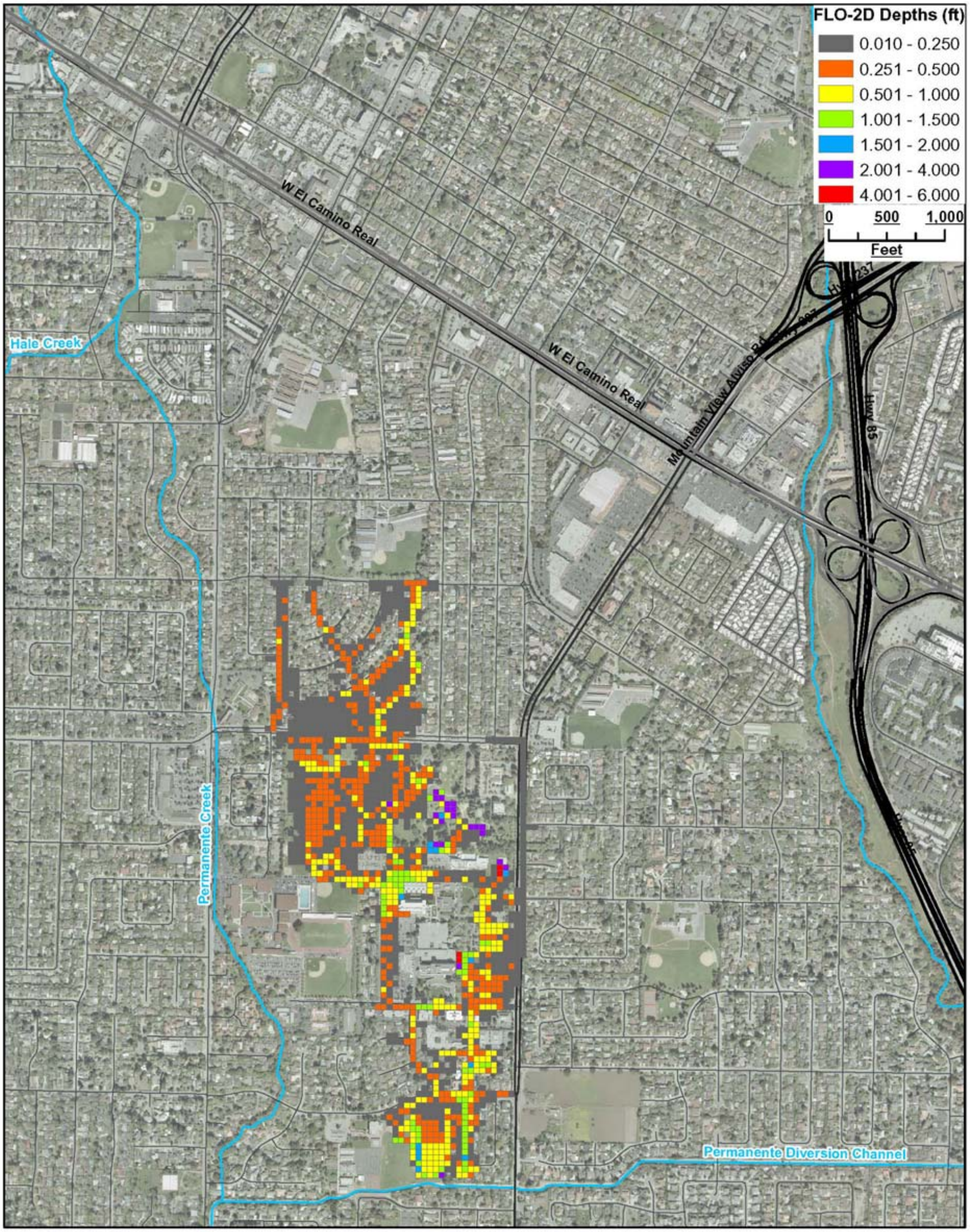


Figure 16: Residual Floodplain with Barbara Avenue Cutoff

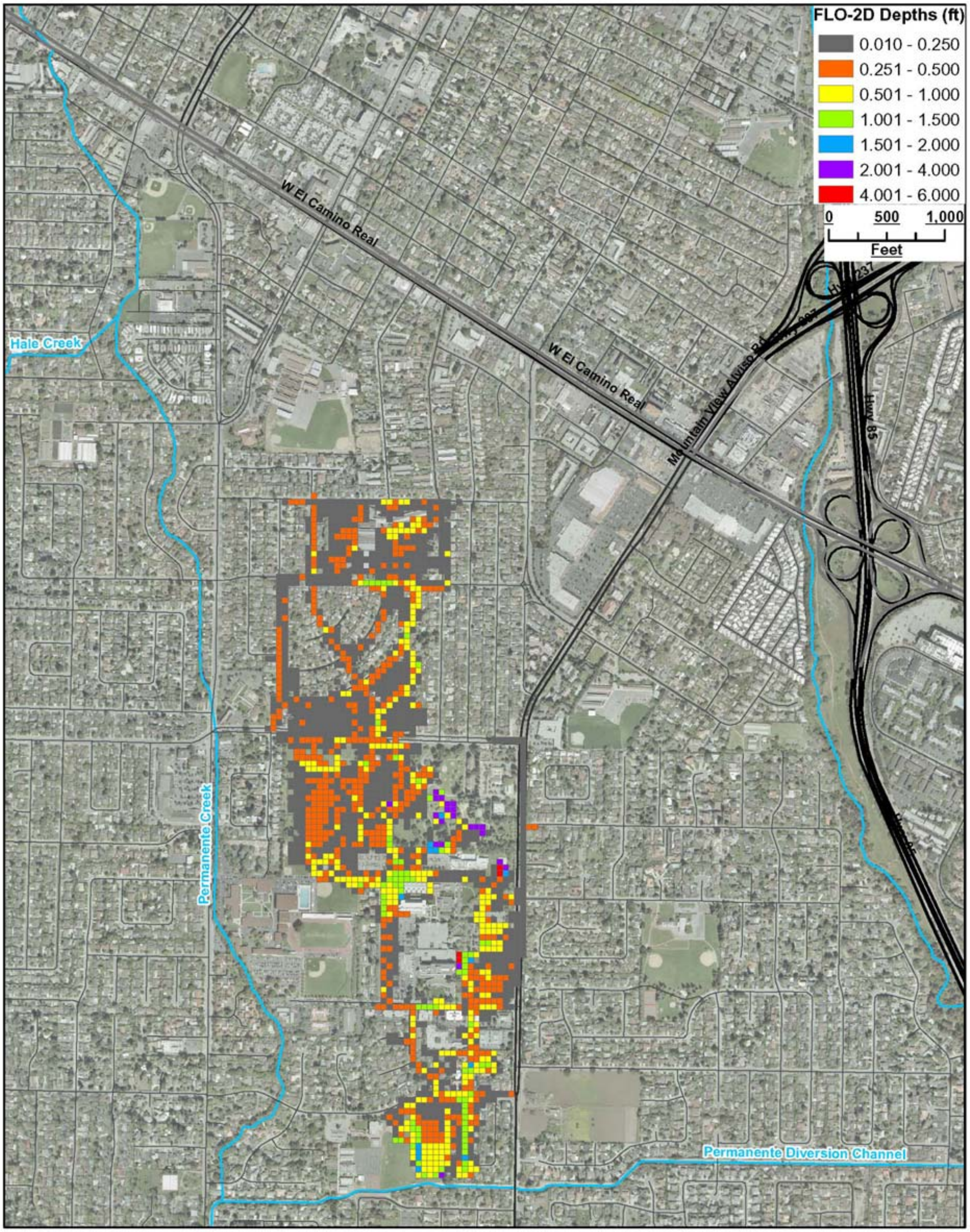


Figure 17: Residual Floodplain with Hans Avenue Cutoff

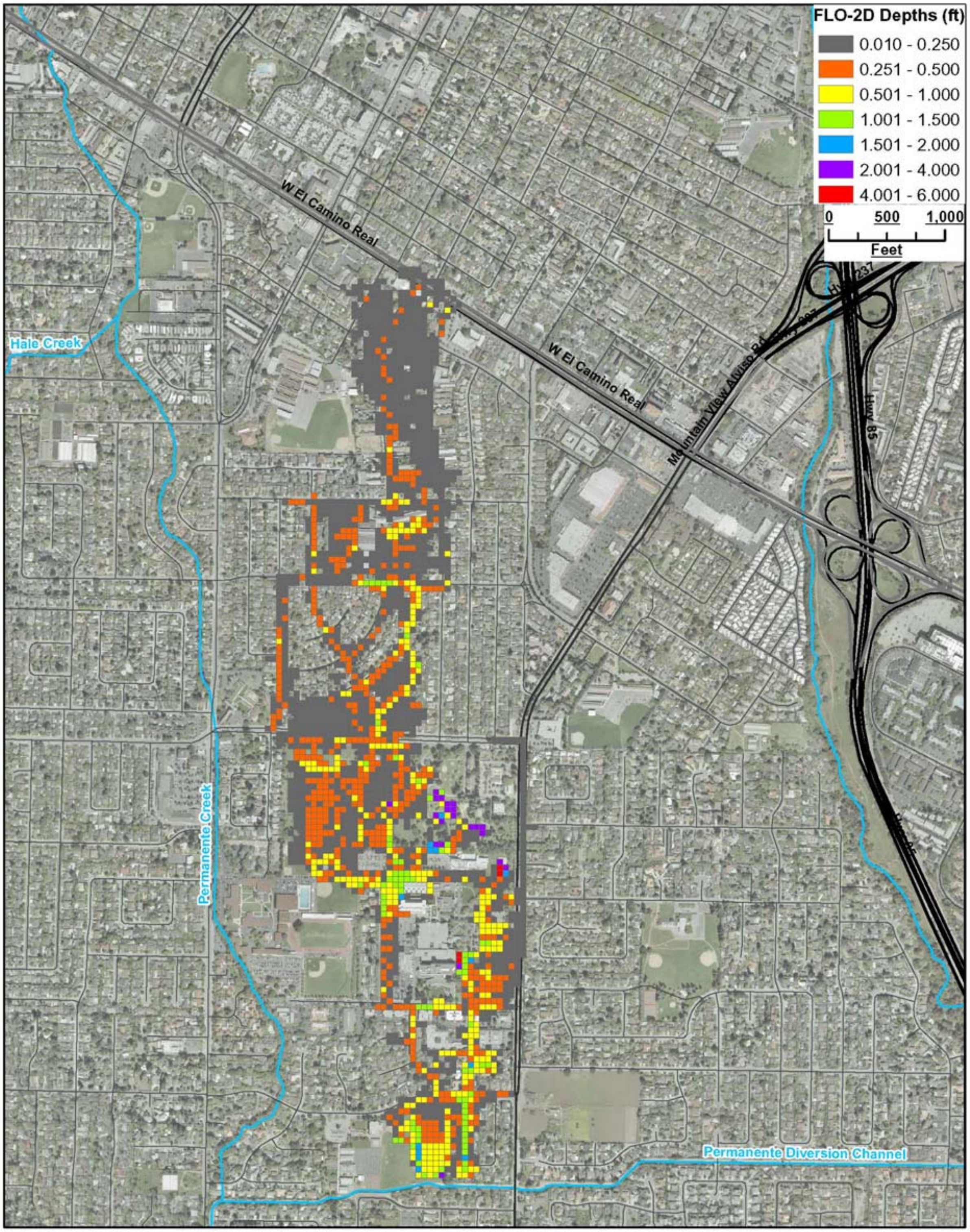


Figure 18: Residual Floodplain with El Camino Real Cutoff

Alternative 2: Miramonte Avenue Bypass

The most direct hydraulic alternative is to bypass the constricted reach of Permanente Creek between Cuesta Drive and Marilyn Drive. In this alternative, the District would design their new diversion structure to send up to about 280 cfs down Permanente Creek as shown in Figure 19, with the remaining discharge sent to the Permanente Diversion and Stevens Creek. Peak discharge that exceeds the 200 cfs capacity of Permanente Creek between Cuesta Drive and Marilyn Drive would be diverted at Cuesta Drive into a buried pipe installed within the Miramonte Avenue right-of-way, as shown schematically previously in Figure 8.

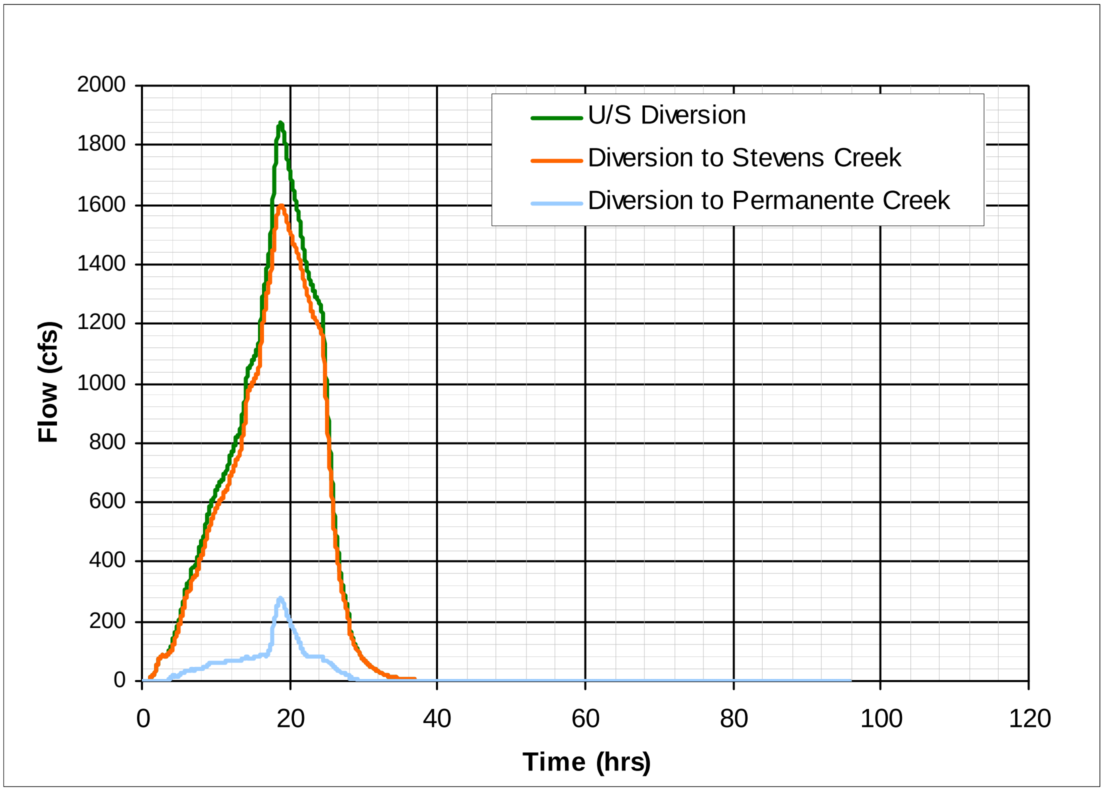


Figure 19: Alternative Flow Split at Permanente Diversion Channel

Flood flow hydrographs are routed through the Miramonte Avenue bypass pipe using the unsteady function of HEC-RAS. The estimated peak discharge in Permanente Creek at Cuesta Drive for this alternative is about 350 cfs, which includes approximately 70 cfs of local stormwater runoff based on the District's HEC-HMS model and the area tributary at Cuesta Drive relative to the total local area tributary to Permanente Creek at the Hale Creek confluence.

A new bifurcation structure would need to be located within the existing footprint of the Permanente Creek culvert in Miramonte Avenue near Cuesta Drive. Somewhere within the approximately 15-foot wide culvert footprint, a 5.5 feet by 5.5 feet control section would limit

Permanente Creek discharge through Cuesta Drive to about 195 cfs, while an adjacent 60-inch diameter pipe would carry excess flow around the creek capacity bottleneck. Detailed civil design and buried utility research are required to ascertain the technical feasibility and relative cost of this alternative, but it is hydraulically viable.

Table 11 summarizes post-project flow routing for this alternative along Permanente Creek from the Diversion downstream to the confluence with Hale Creek near McKelvey Park, and along the Permanente Diversion from Permanente Creek to Stevens Creek, assuming that the existing choke structure within the Diversion Channel near Blach Intermediate School is removed. Figure 20 shows Permanente Creek hydrograph routing for this alternative.

Table 11
Flow Routing of Base Flood Event from Permanente Diversion to McKelvey Park

Creek/Reach/Location	Design Capacity (cfs)	Existing 1% Flow (cfs)	Design 1% Discharge (cfs)	Bypass Flow (cfs)
PERMANENTE CREEK				
At Confluence with Permanente Diversion	2,100	2,417	1,878	283
Diversion at Cuesta Drive	500	70	353	158
Cuesta Drive to Barbara Avenue	250	70	195	-
Barbara Avenue to Marilyn Drive	200	70	195	-
Marilyn Drive to Hale Creek Confluence	500	110	392	-
Hale Creek Confluence to McKelvey Park	1,500	1,184	1,447	-
PERMANENTE DIVERSION				
Permanente Creek to Blach School	1,600	2,417	1,600	-
Blach School Spill	-	817	-	-
Blach School to Stevens Creek	1,600	1,600	1,600	-

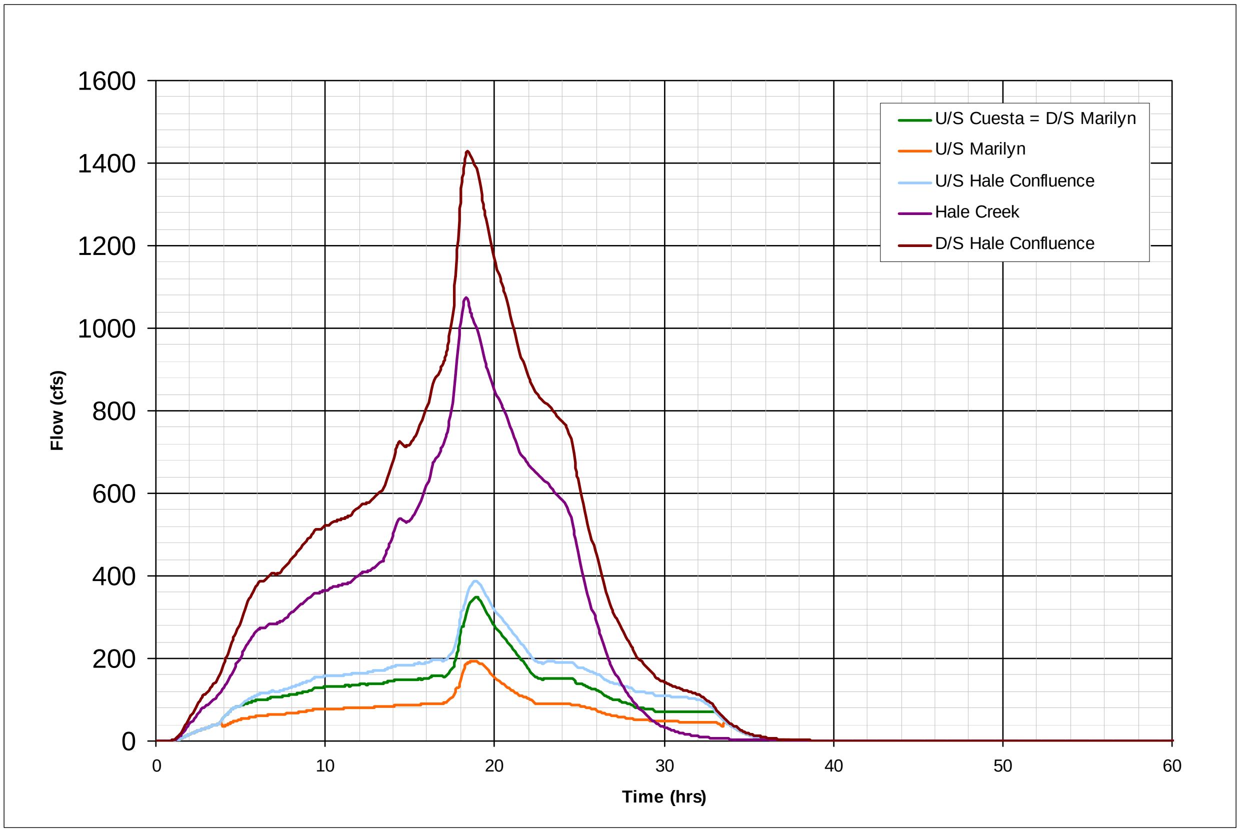


Figure 20: Hydrograph Routing to McKelvey Park (Alternative 2)

Cuesta Park Annex

The proposed Cuesta Park Annex storage basin would be excavated to a depth that allows gravity drainage to Permanente Creek without the need for a long drain line along Miramonte Avenue to Marilyn Drive. Given these constraints the maximum size required for the Cuesta Park Annex basin would be about 8 feet deep and store up to 34 acre-feet of diverted runoff. Conceptual drawings and details for various Cuesta Park Annex alternatives may be found in Appendix B.

The actual mechanics of diverting excess Permanente Creek flow into the storage basin for attenuation and release depend upon the selected diversion alternative.

Alternative 1a – Intercept Residual Floodplain at Cuesta Drive

If the intercepted flow is returned directly to Permanente Creek at Cuesta Drive, the combined routed peak discharge is about 210 cfs, which would exceed the capacity of the creek channel between Cuesta Drive and Marilyn Avenue. Therefore some flood storage within the Cuesta Annex Basin would be required for Alternative 1a, to sufficiently reduce the flow in Permanente Creek (to less than 200 cfs) for safe conveyance to the Hale Creek confluence.

To accomplish this, the Cuesta Park Annex storage basin needs a total storage volume of 9 acre-feet with a maximum excavated depth of 2 feet, using the storage-elevation data for the basin presented in Table 12, and the corresponding storage-elevation curve (Figure 21).

Table 12
Storage-Elevation Data for Cuesta Park Annex Basin
(Alternative 1a)

Cuesta Park Annex	
Elevation (ft)	Storage (ac-ft)
139	0.00
140	4.39
141	8.92

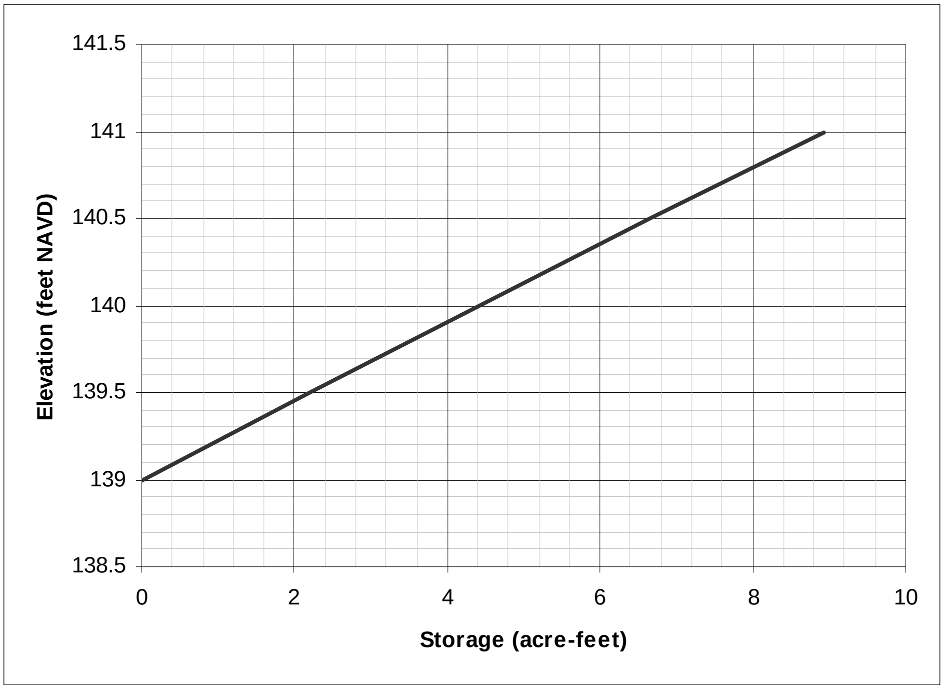


Figure 21: Storage-Elevation Curve for Cuesta Annex Basin (Alternative 1a)

Outflow from the Cuesta Drive cutoff drain would enter the Cuesta Annex Basin via the 54-inch diameter interceptor pipe, through an inlet located at the northeast corner of the basin. For public warning and safety, a bubble-up discharge system with an emergency inlet above the basin bottom is proposed (Appendix B). Stored water would be drained from the basin through a 24-inch diameter pipe with a screened inlet located at the northwest corner of the basin, which would return stored water back to Permanente Creek on the downstream side of the existing Miramonte Avenue culvert at Cuesta Drive, where the approximate invert is elevation 130 feet, as shown in Appendix B.

Table 13 summarizes post-project flow routing for this alternative along Permanente Creek from the Diversion downstream to the confluence with Hale Creek and along the Permanente Diversion from Permanente Creek to Stevens Creek, assuming that the existing choke structure within the Diversion Channel near Blach Intermediate School is removed.

Table 13
Flow Routing with Residual Floodplain at Blach Intermediate School (Alternative 1a)

Creek/Reach/Location	Design Capacity (cfs)	Existing 1% Flow (cfs)	Design 1% Discharge (cfs)	Inflow (cfs)	Return Flow (cfs)	Target Storage (ac-ft)
PERMANENTE CREEK						
At Confluence with Permanente Diversion	2,100	2,417	1,878			
Cuesta Drive to Barbara Avenue	250	70	170			
Barbara Avenue to Marilyn Drive	200	70	170			
Marilyn Drive to Hale Creek Confluence	500	110	210			
Hale Creek Confluence to McKelvey Park	1,500	1,184	1,267			
PERMANENTE DIVERSION						
Permanente Creek to Blach School	1,600	2,417	1,600			
Blach School Spill	-	817	817			
Blach School to Stevens Creek	1,600	1,600	1,600			
Floodplain at Cuesta Drive	-	-	60			
Cuesta Park Annex	-	-	-	60	5	9

Alternative 3: Divert Excess Flow at Cuesta Drive

The facilities at Cuesta Drive would be similar to those proposed for Alternative 1a. However, rather than limit upstream diversions to 100 cfs and allow 100-year spill from the Permanente Diversion at Blach Intermediate School, the 280 cfs diversion of Alternative 1 would be restored to eliminate the residual floodplain between Blach School and Cuesta Drive, and the peak discharge at the Cuesta Drive diversion is 350 cfs as for Alternative 2. In lieu of the Miramonte Bypass from Alternative 2, however, the Cuesta Park Annex storage basin would be constructed to reduce downstream discharge to within the restricted channel capacity limits between Cuesta Drive and Marilyn Drive, with high flows diverted from Permanente Creek at Cuesta Drive.

The simplest diversion would be similar to the return pipe connection from Alternative 1a, although it would need to be a larger pipe to connect Permanente Creek to the Cuesta Park Annex storage basin. A diversion could consist of cutting into the existing 14′-3″ by 8′-11″ corrugated metal pipe arch and building a concrete junction structure. However, hydraulic analyses using unsteady state models indicate that a simple bifurcation of flow at the Cuesta diversion structure would require substantially more storage volume, because early in the storm flow enters the basin and consumes available storage before it is needed when the peak of the flood hydrograph passes.

To avoid more frequent inundation within the Cuesta Park Annex storage basin and a premature loss of storage for storm events, the Cuesta Drive diversion needs to be in the form of an overflow weir designed to prevent spill to Cuesta during all but the most extreme runoff events. As shown in Appendix B, there are two potential locations for this overflow weir. The design of the overflow weir does affect the required size of storage basin, as tabulated in Table 14 and shown graphically in Figure 22.

Table 14
Storage-Elevation Data for Cuesta Annex Basin (Alternative 3)

ALTERNATIVE 3A		ALTERNATIVE 3B	
Elevation (ft)	Storage (ac-ft)	Elevation (ft)	Storage (ac-ft)
131	0	132	0
132	6.4	133	3.4
133	10.0	134	7.0
134	13.7	135	10.7
135	17.6	136	14.6
136	21.7	137	18.7
137	25.9	138	22.9
138	30.3	139	27.3
139	33.8	140	31.8

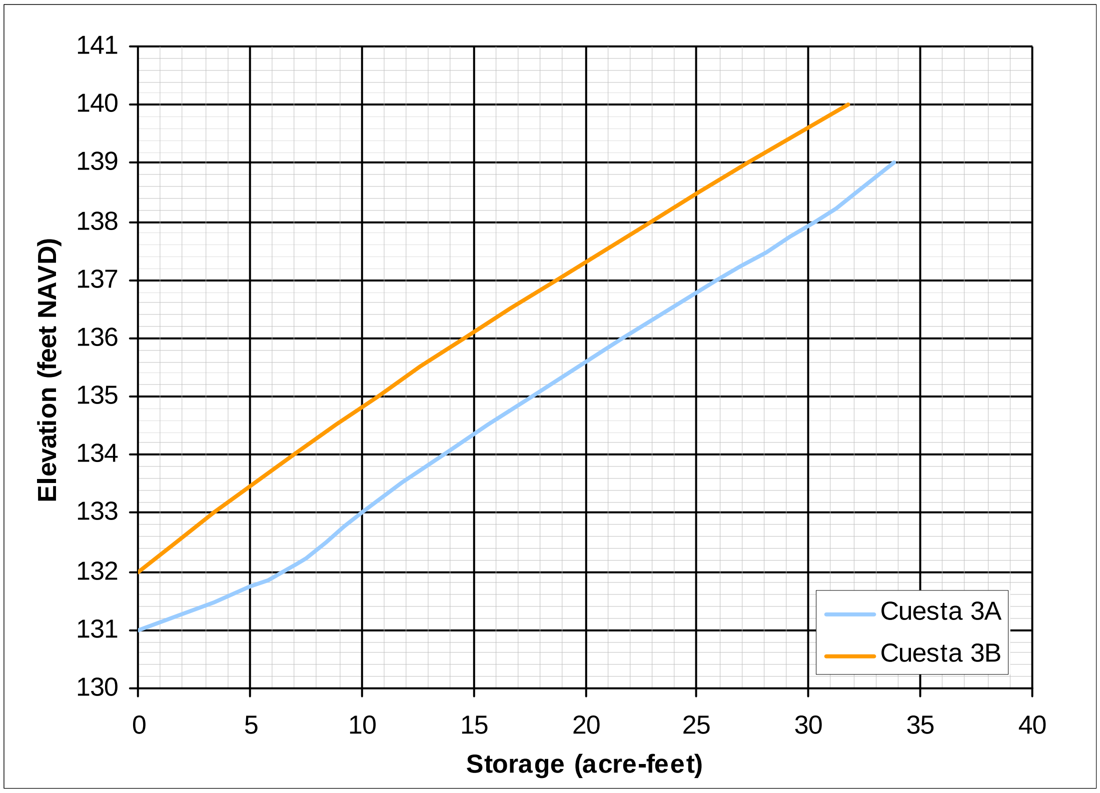


Figure 22: Storage-Elevation Curves for Cuesta Park Annex (Alternative 3)

Alternative 3a – Underground Diversion Structure

An underground diversion structure would be created at the intersection of Cuesta Drive and Miramonte Avenue by cutting into the existing 14′-3″ by 8′-11″ corrugated metal pipe arch and building a concrete junction structure with an internal 50-foot long weir set at elevation 137.9 feet. Within the downstream wall of the structure a 5.5 feet by 5.5 feet opening would be formed to restrict downstream flow to 195 cfs. Excess flow would be forced over the weir and into the Cuesta Park Annex through a 1,200 feet long interconnecting pipe. Basin drainage would be accomplished via a flapgated 12-inch outfall to the downstream side of the Miramonte Avenue culvert.

Figure 23 illustrates how the overflow basin would perform under this alternative, which is also summarized in Table 15. It is noted that the basin empties after more than 48 hours from first spill. To avoid vector control problems with the small amount of local overland storm water runoff that would enter the basin, a permeable layer that allows long-term infiltration through the bottom of the basin is recommended.

Alternative 3b – Diversion Structure in Creek Channel

Alternatively, an open diversion structure would be constructed within the Permanente Creek channel right-of-way immediately upstream from the Cuesta Drive culvert inlet. The requisite 100-foot long weir would be set at elevation 139.5 feet. A culvert restriction formed with a 4.75 feet by 4.75 feet opening would limit downstream flow to 195 cfs. Excess flow would be forced over the weir into a receiving channel and into the Cuesta Park Annex through a similar interconnecting pipe. Basin drainage would be accomplished via a flapgated 18-inch diameter return opening from the receiving channel to the Permanente Creek culvert on the downstream side of Miramonte Avenue.

Figure 23 illustrates how the overflow basin would perform under this alternative, compared to Alternative 3a, both of which are also summarized in Table 15. It is noted that the basin empties after more than 48 hours from first spill. The difficulty of constructing an open channel weir, considering limited right-of-way, the proximity of structures and overhead utilities, and regulatory permitting requires further evaluation.

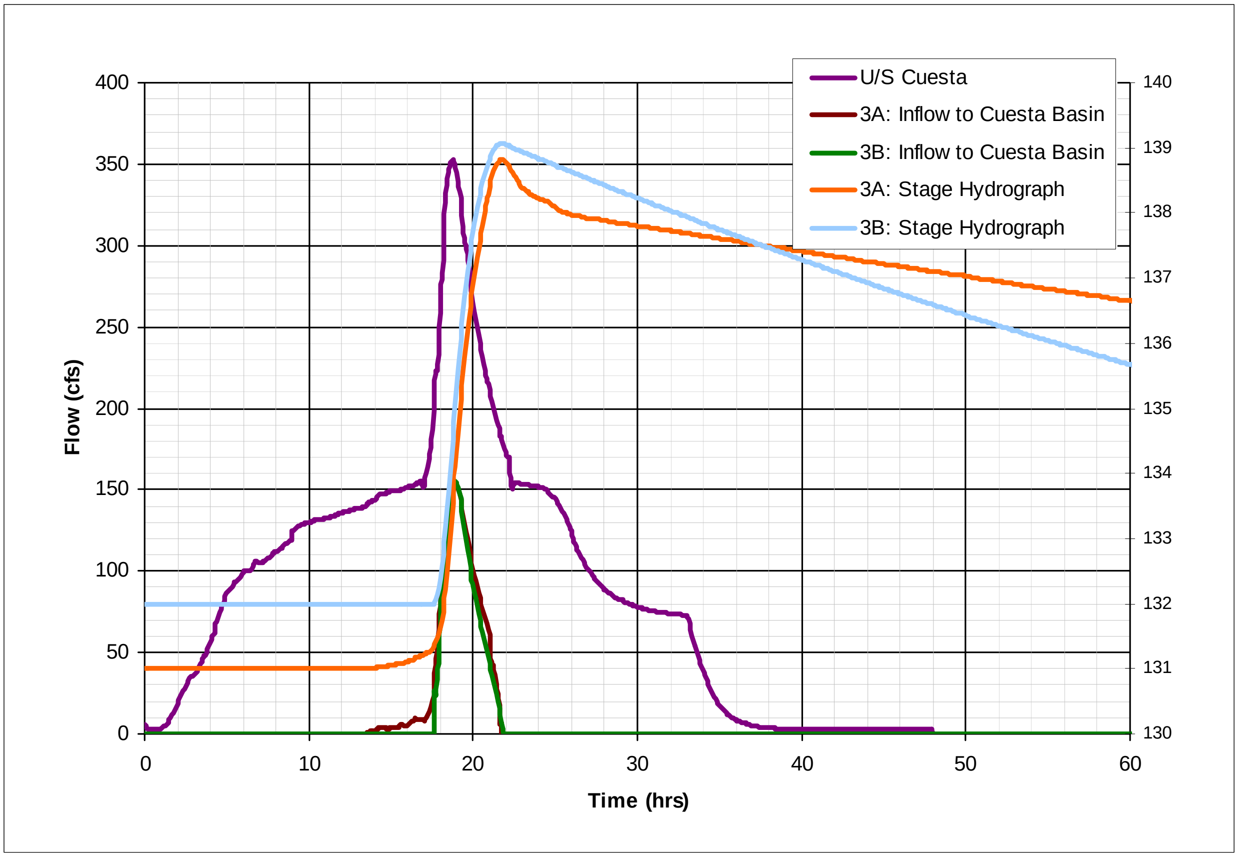


Figure 23: Flow Diversion at Cuesta Park Annex

Table 15
Summary of Cuesta Park Annex Basin Flood Routing with Overflow Weir

Routing Parameter	**Alternative 3a Underground Weir**	**Alternative 3b Channel Weir**
CONTROL STRUCTURE		
Peak Discharge on Permanente Creek at Cuesta Drive	353 cfs	353 cfs
Peak Discharge through Control Structure	195 cfs	183 cfs
Peak Discharge to Cuesta Park Annex Storage Basin	158 cfs	170 cfs
Peak Recombined Flow Downstream on Permanente Creek	195 cfs	193 cfs
CUESTA PARK ANNEX STORAGE BASIN		
Inflow Volume	33.4	30.6
Retained Volume	33.4	30.6
Return Volume	33.4	30.6
Maximum Elevation in Basin	138.7	139.7
Peak Return Flow	2	13
Time to Drain	>48 hrs	>48 hrs

Table 16 details the post-project flow routing along Permanente Creek from the Permanente Diversion downstream to the confluence with the Hale Creek and on to McKelvey Park, assuming Alternative 3a, which is also shown graphically as Figure 24.

Table 16
Flow Routing from Permanente Diversion to McKelvey Park (Alternative 3a)

Creek/Reach/Location	Design Capacity (cfs)	Existing 1% Flow (cfs)	Design 1% Discharge (cfs)	Inflow (cfs)	Return Flow (cfs)	Target Storage (ac-ft)
PERMANENTE CREEK						
Permanente Diversion to Cuesta Drive	500	70	353	-	-	-
Cuesta Park Annex Basin	-	-	-	158	2	33.4
Cuesta Drive to Barbara Avenue	250	70	195	-	-	-
Barbara Avenue to Marilyn Drive	200	70	195	-	-	-
Marilyn Drive to Hale Creek Confluence	500	110	235	-	-	-
Hale Creek Confluence to McKelvey Park	1,500	1,184	1,305	-	-	-

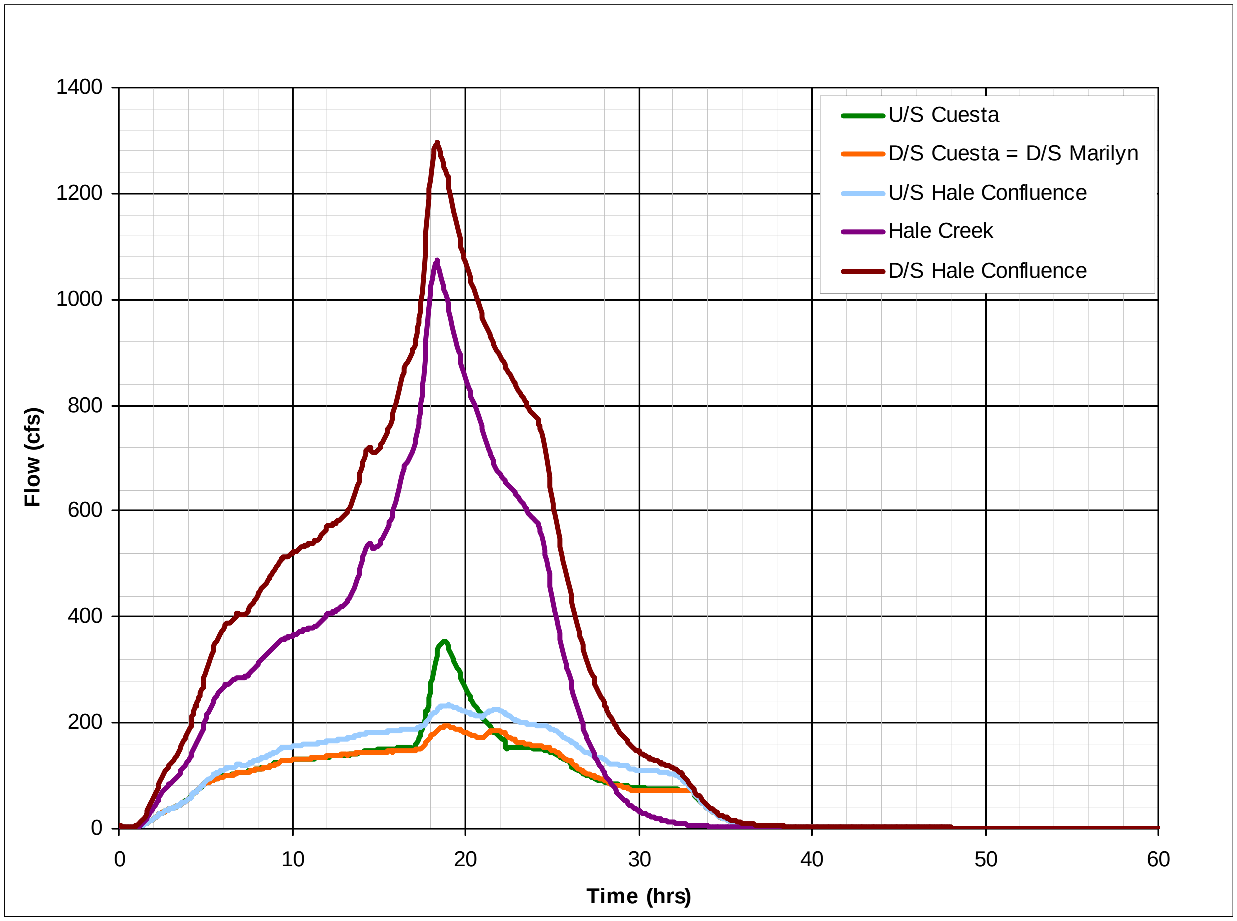


Figure 24: Flow Routing from Permanente Diversion to McKelvey Park

McKelvey Park

The McKelvey Park storage basin receives flood flows that exceed the downstream capacity of Permanente Creek. Rough storage-elevation data for a potential basin at McKelvey Park are presented in Table 17, and the corresponding storage-elevation curves are shown in Figure 25.

Table 17
Storage-Elevation Data for McKelvey Park Basin

McKelvey Park	
Elevation (ft)	**Storage (ac-ft)**
90	0.00
105	59.24

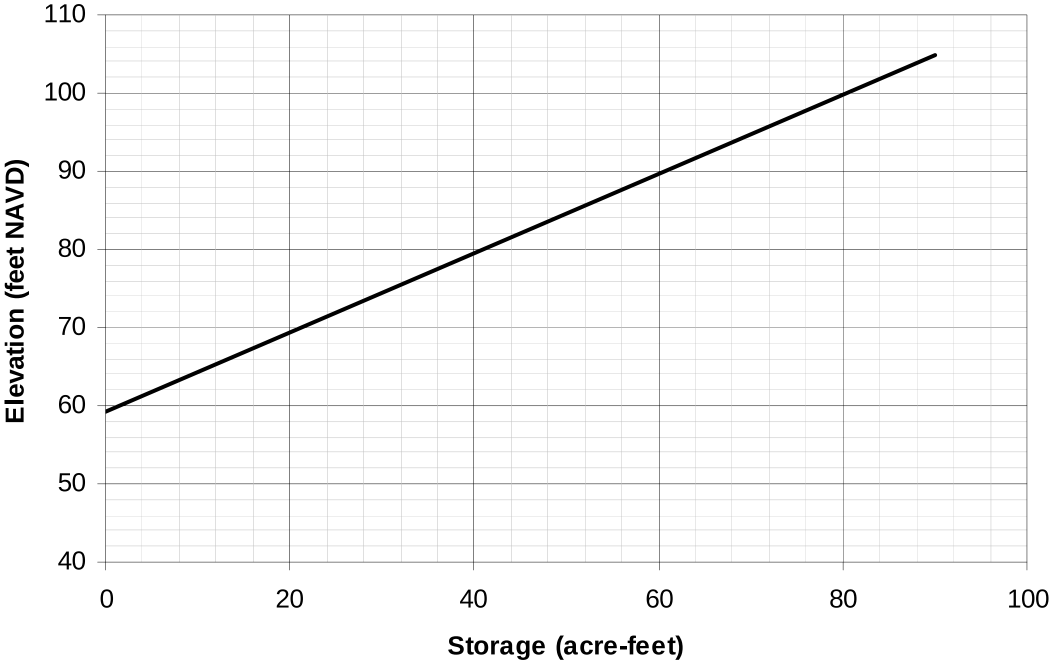


Figure 25: Storage-Elevation Curve for McKelvey Park Basin

Excess flow is diverted to the park via a 200-foot long lateral weir at elevation 108.55 feet NAVD, located on the east side of Permanente Creek just upstream of the Mountain View Avenue Bridge. The bridge acts as a hydraulic control, limiting downstream flows. Water flowing over the weir will fall into a 6-foot wide side channel that directs the overflow back into an open control structure that releases water into McKelvey Park. Open grating is proposed to cover the structure.

Stored water will drain from the park through a 54-inch diameter pipe located at the northwest corner of the park and will flow back into Permanente Creek. To realize gravity drainage from the newly depressed McKelvey Park, this new storm drain needs to outfall to the creek at an elevation no higher than the bottom of the excavated park. (A dewatering sump pump may still be required for local park drainage and any subsurface drainage system that is installed.) The City of Mountain View currently owns a 60-inch diameter storm drain in nearby Park Avenue. However the available capacity in this drainage facility is not sufficient to accommodate 100-year McKelvey Park return flows without causing backup and flooding on El Camino Real and adjacent streets.

Table 18 summarizes hydraulic analyses that establish the required storage volume within McKelvey Park based on alternative upstream diversions. At a maximum elevation of 105 feet, the basin would store 59 acre-feet of storm water. The McKelvey Park basin would empty by gravity within 12 hours of the first overflow from Permanente Creek for all upstream diversion alternatives other than the Miramonte Bypass Alternative, which requires 48 hours to drain the basin. This performance is well within Santa Clara County's general requirement that detained water be drained within three to five days for vector control.

Table 18
Summary of McKelvey Park Flood Routing

Routing Parameter	Upstream Diversion Alternative						
	1a.	1b.	1c.	1d.	2.	3a.	3b.
CONTROL STRUCTURE at Mountain View Avenue							
Peak Upstream Discharge in Permanente Creek (cfs)	1,268	-	1,268	1,268	1,447	1,305	1,305
Peak Discharge thru Mountain View Ave Bridge (cfs)	931	-	931	931	980	943	943
Peak Discharge to McKelvey Park Basin (cfs)	336	-	336	336	466	363	363
STORAGE BASIN							
Inflow Volume (ac-ft)	86.7	-	86.7	86.7	115.4	92.7	92.7
Retained Volume (ac-ft)	56.4	-	56.4	56.4	59.0	59.0	59.0
Volume Released (ac-ft)	86.7	-	86.7	86.7	115.4	92.7	92.7
Maximum Elevation in Basin (feet NAVD)	104.2	-	104.2	104.2	104.9	105.0	105.0
Peak Flow to Drain (cfs)	113	-	113	113	176	120	120
Location of Outfall for Gravity Drainage†	PD	-	PD	PD	ECR	PD	PD
Time to Drain (hours)	12	-	12	12	48	12	12

1a. Intercept Blach Spill at Cuesta Drive
1b. Intercept Blach Spill at Barbara Avenue (infeasible)
1c. Intercept Blach Spill at Hans Avenue
1d. Intercept Blach Spill at El Camino Real
†PD = Park Drive; ECR = El Camino Real

2. Miramonte Avenue Bypass
3a. Underground Diversion Weir at Cuesta Drive
3b. Diversion Weir in Channel Upstream of Cuesta Drive

PROJECT PERFORMANCE SUMMARY

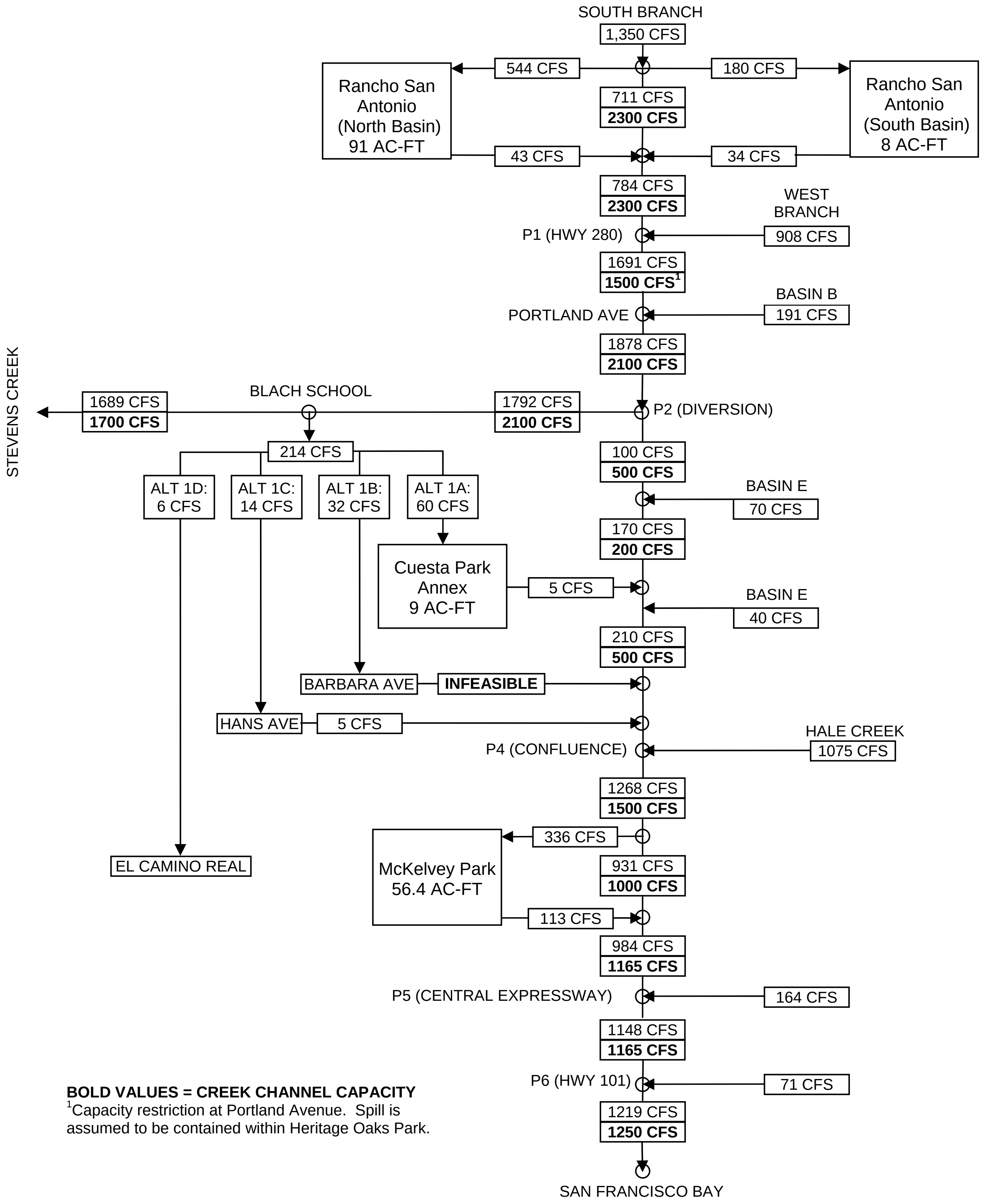
Although there are nuanced differences in project performance depending upon the selected Permanente Creek diversion and Cuesta Park Annex storage basin alternatives, each alternative presented herein would meet the stated Project objective, which is to remove the special flood hazard zone north of El Camino Real. Without presenting a recommendation, since additional civil and geotechnical engineering are required, a suite of improvements is analyzed for overall system performance assuming Alternative 3a is selected to illustrate post-project performance, which is summarized by Table 19 for the one-percent design event. Figures 26, 27, and 28 schematically detail base flood routing for Diversion Alternatives 1, 2, and 3 respectively.

Table 19
Flow Routing from Rancho San Antonio to San Francisco Bay

Creek/Reach/Location	Design Capacity (cfs)	Existing 1% Flow (cfs)	Design 1% Discharge (cfs)	Inflow (cfs)	Return Flow (cfs)	Target Storage (ac-ft)
PERMANENTE CREEK						
Upstream of Rancho San Antonio Park	n/a	1,350	1,350	-	-	-
North Rancho San Antonio Park Basin	-	-	-	724	77	100
Rancho San Antonio to Foothill Expressway	2,300	2,232	1,691	-	-	-
Foothill Expressway to Portland Avenue[1]	1,500	2,232	1,691	-	-	-
Portland Avenue to Diversion	2,100	2,417	1,878	-	-	-
Diversion to Cuesta Drive	500	70	353	-	-	-
Cuesta Park Annex Basin	-	-	-	158	2	33.4
Cuesta Drive to Hale Creek	200	110	195	-	-	-
Hale Creek to McKelvey Park	1,500	1,184	1,305	-	-	-
McKelvey Park Basin	-	-	-	363	120	59
McKelvey Park to El Camino Real[2]	1,000	600	1,000	-	-	-
El Camino Real to Central Expressway	1,000	600	1,000	-	-	-
Central Expressway to Bayshore Freeway	1,165	764	1,163	-	-	-
Bayshore Freeway to San Francisco Bay	1,250	835	1,234	-	-	-
PERMANENTE DIVERSION						
Permanente Creek to Blach School	1,600	2,417	1,600	-	-	-
Blach School Spill	-	817	0	-	-	-
Blach School to Stevens Creek	1,600	1,600	1,600	-	-	-
Diversion to Permanente Creek	285	0	285	-	-	-

[1] Portland Avenue Bridge assumed to be replaced per City of Los Altos plans. Capacity without existing local restriction is 2,100 cfs.

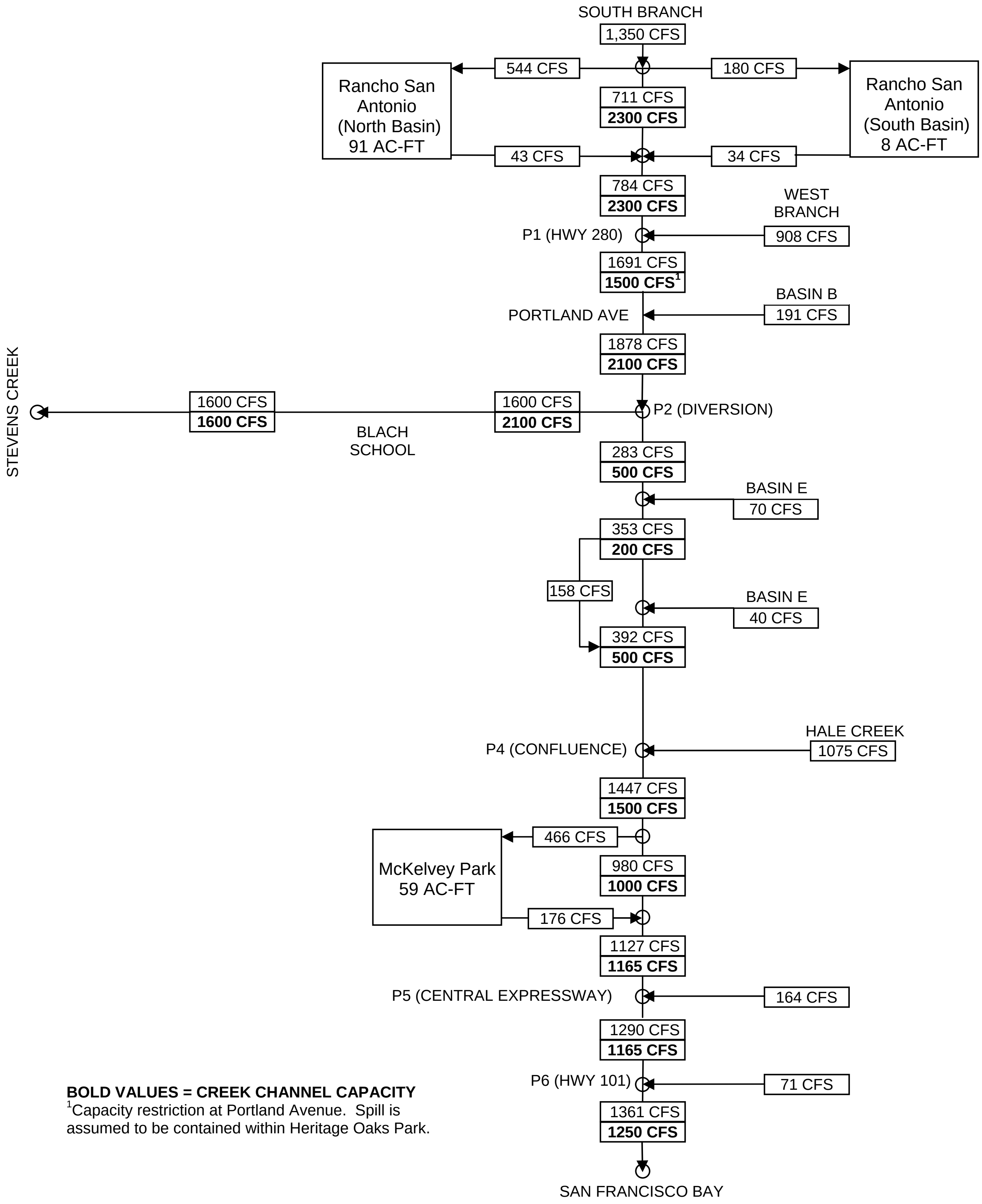
[2] Capacity restriction at footbridge downstream of Park Avenue. Capacity without local restriction is 1,000 cfs.



BOLD VALUES = CREEK CHANNEL CAPACITY

[1] Capacity restriction at Portland Avenue. Spill is assumed to be contained within Heritage Oaks Park.

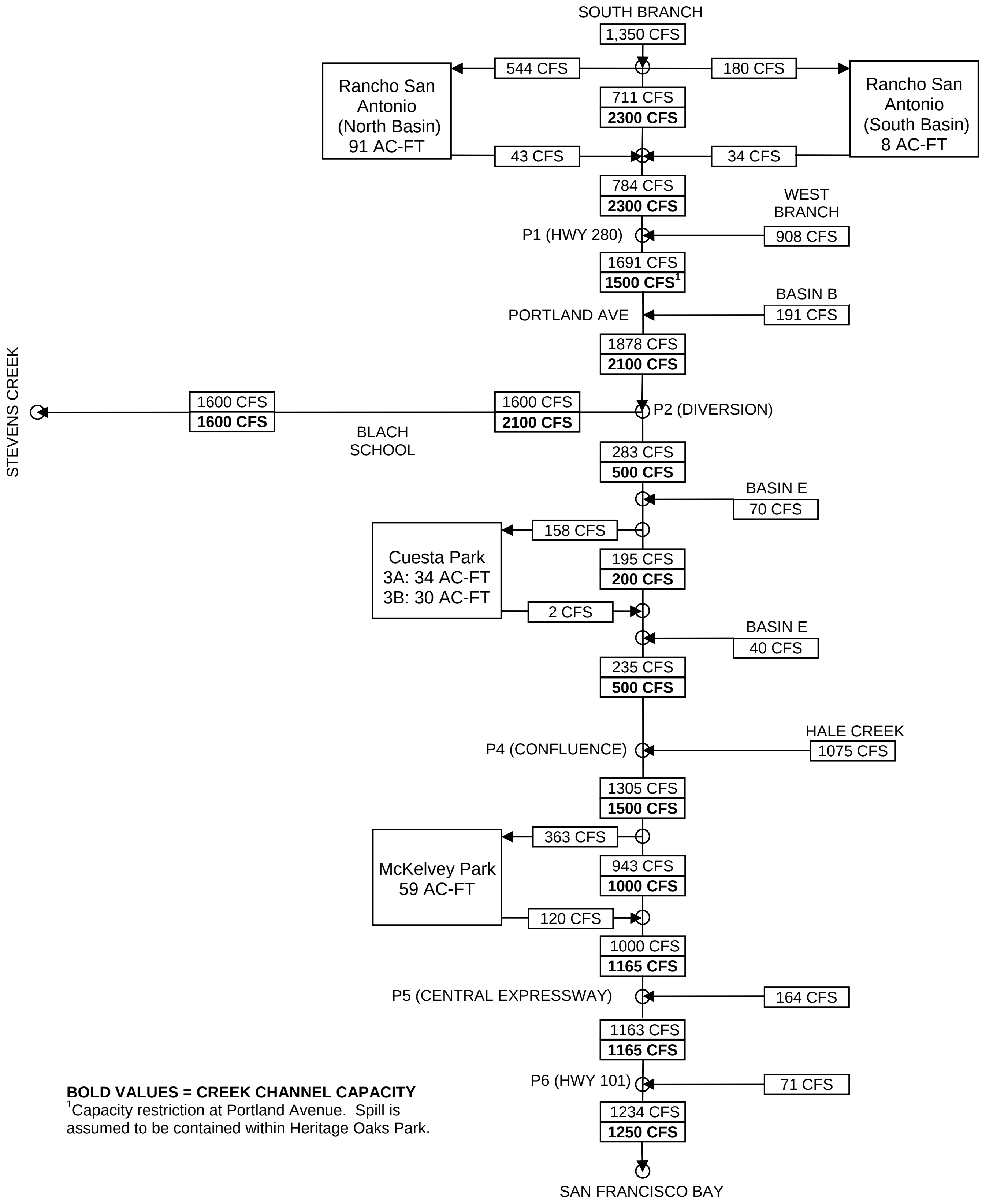
Figure 26: Base Flood Routing Schematic (Diversion Alternative 1)



BOLD VALUES = CREEK CHANNEL CAPACITY

[1] Capacity restriction at Portland Avenue. Spill is assumed to be contained within Heritage Oaks Park.

Figure 27: Base Flood Routing Schematic (Diversion Alternative 2)



BOLD VALUES = CREEK CHANNEL CAPACITY

[1] Capacity restriction at Portland Avenue. Spill is assumed to be contained within Heritage Oaks Park.

Figure 28: Base Flood Routing Schematic (Diversion Alternative 3)

Ten-Year Flood Event

Public outreach programs that have supported the District's 2008 Planning Study have generally indicated that overflows into the various off-stream storage basins would not occur during events more frequent than the ten-year return period. (Table 1 provides a synopsis of the probability of experiencing a ten-year event over various periods of time.) Flows from Table 2 are used to assess post-Project performance during a design ten-year event following the same flood routing methodology for Alternative 3a, and are summarized by Table 20.

Table 20
Post-Project Summary of Ten-Year Event

Creek/Reach/Location	Existing 10% Flow (cfs)	Design 10% Discharge (cfs)	Inflow (cfs)	Return Flow (cfs)	Maximum Flood Depth (feet)	Time to Drain (hours)
PERMANENTE CREEK						
Upstream of Rancho San Antonio Park	757	757	-	-	-	-
Rancho San Antonio Park Basins	-	-	63	1	0.5	*
Rancho San Antonio to Foothill Expressway	1,186	1,103	-	-	-	-
Foothill Expressway to Portland Avenue	1,303	1,218	-	-	-	-
Portland Avenue to Diversion	1,303	1,218	-	-	-	-
Diversion to Cuesta Drive	154	152	-	-	-	-
Cuesta Park Annex Basin	-	-	6	≈ 0	0.1	*
Cuesta Drive to Hale Creek	146	184	-	-	-	-
Hale Creek to McKelvey Park	811	809	-	-	-	-
McKelvey Park Basin	-	-	18	≈ 0	0.15	-
McKelvey Park to El Camino Real	811	791	-	-	-	-
El Camino Real to Central Expressway	811	791	-	-	-	-
Central Expressway to Bayshore Freeway	972	970	-	-	-	-
Bayshore Freeway to San Francisco Bay	1,041	1,040	-	-	-	-
PERMANENTE DIVERSION						
Permanente Creek to Blach School	1,220	1,135	-	-	-	-
Blach School to Stevens Creek	1,220	1,135	-	-	-	-
Diversion to Permanente Creek	1,220	1,135	-	-	-	-

*Anticipate drainage through infiltration

The facility most sensitive to ten-year flooding is Rancho San Antonio Park which receives less 63 cfs in overflow during the ten-year event. At Rancho San Antonio, water accumulates to a depth of approximately six inches during the ten-year event. This would either drain by gravity or via infiltration through the basin soil. If drained solely by gravity, the resulting head in the basin means the remaining six inches of water would very drain slowly over the course of many das. It is presumed that the basin floor will be permeable and allow for more rapid dewatering through soil infiltration.

Potential Project Phasing

With the exception of modifications to the Permanente diversion, which presently sends no flow down the natural creek channel, any of the flood protection facilities described herein would only improve the flooding situation. That is, constructing any one of the storage facilities would not worsen local flooding by either inducing flooding where there is presently no flooding or increasing the depth of flooding that would have occurred but for the construction of a given storage basin.

Before flow is restored to the natural Permanente Creek channel by reconstructing the diversion pipe, the McKelvey Park basin should be built, and if the diversion structure is built to deliver 280 cfs to the creek channel, the Cuesta Park Annex Basin or Miramonte Avenue Bypass basin must be completed prior to diverting the full flow. Otherwise there is a potential to cause flooding within the restricted channel reach between Cuesta Drive and Marilyn Drive.

SELECTED REFERENCES

1. Amer. Sos. Civil Engrs., Sedimentation Engineering, ASCE Press, New York, 1995.

2. Chow, V.T., Open Channel Hydrualics, McGraw-Hill, New York, 1959.

3. Santa Clara Valley Water District, “Hydrology Report of the Permanente Creek Watershed,” November 2007.

4. Federal Emergency Management Agency, National Flood Insurance Program Regulations, 44 CFR Parts 59-78, May 1998.

5. Henderson, F.M., Open Channel Flow, MacMillan, New York, 1966.

6. Natural Resources Conservation Service, Engineering Field Handbook, Chapter 16: “Streambank and Shoreline Protection,” 1996.

7. Santa Clara Valley Water District, “Engineering Policies and Procedures Memorandum re: Freeboard,” September 16, 1994.

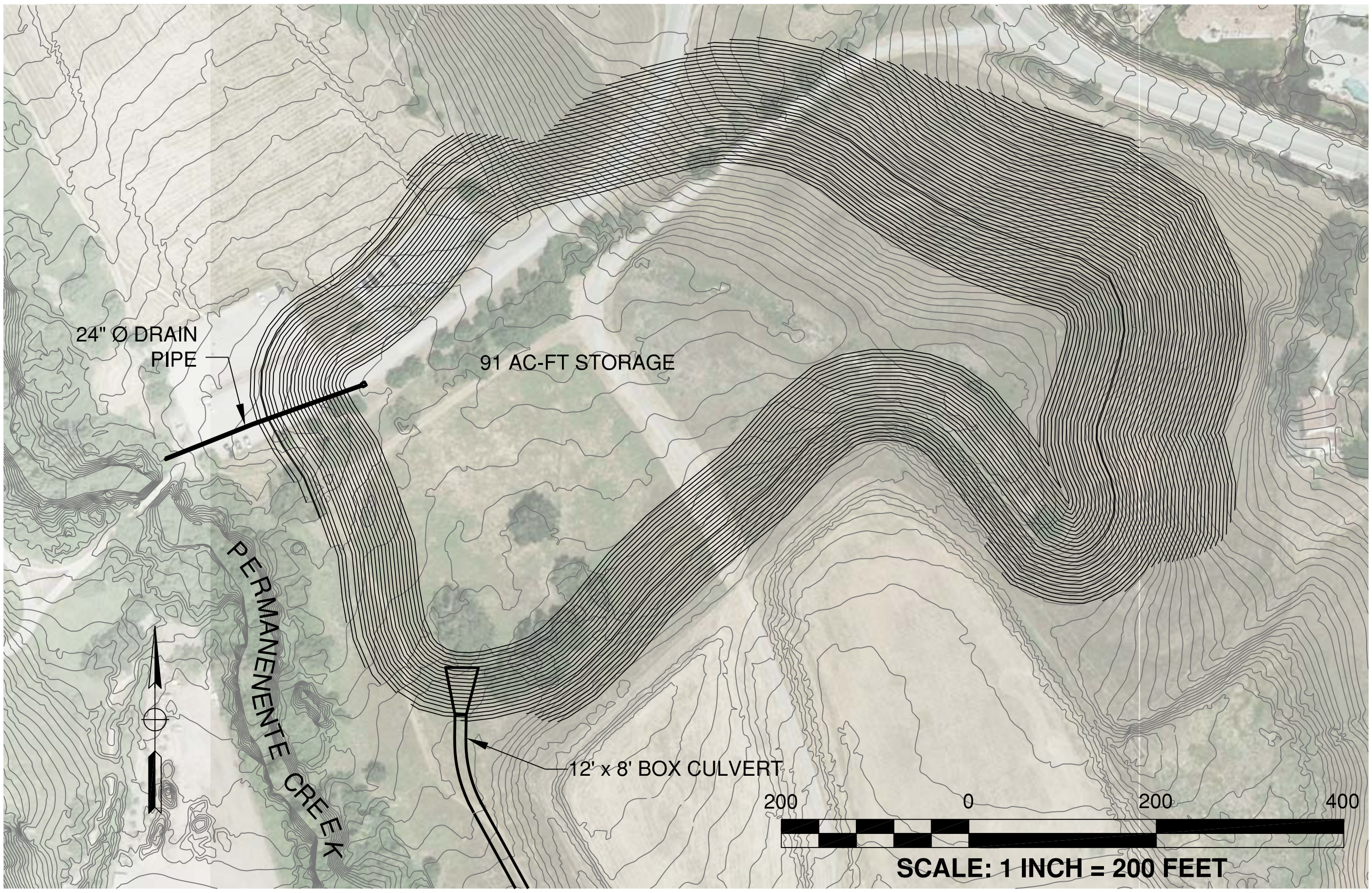
8. U.S. Army Corps of Engineers, Hydraulic Design of Flood Control Channels, EM 1110-2-1601, 1994.

9. ibid, Accuracy of Computed Water Surface Profiles, FHA RD26, 1986.

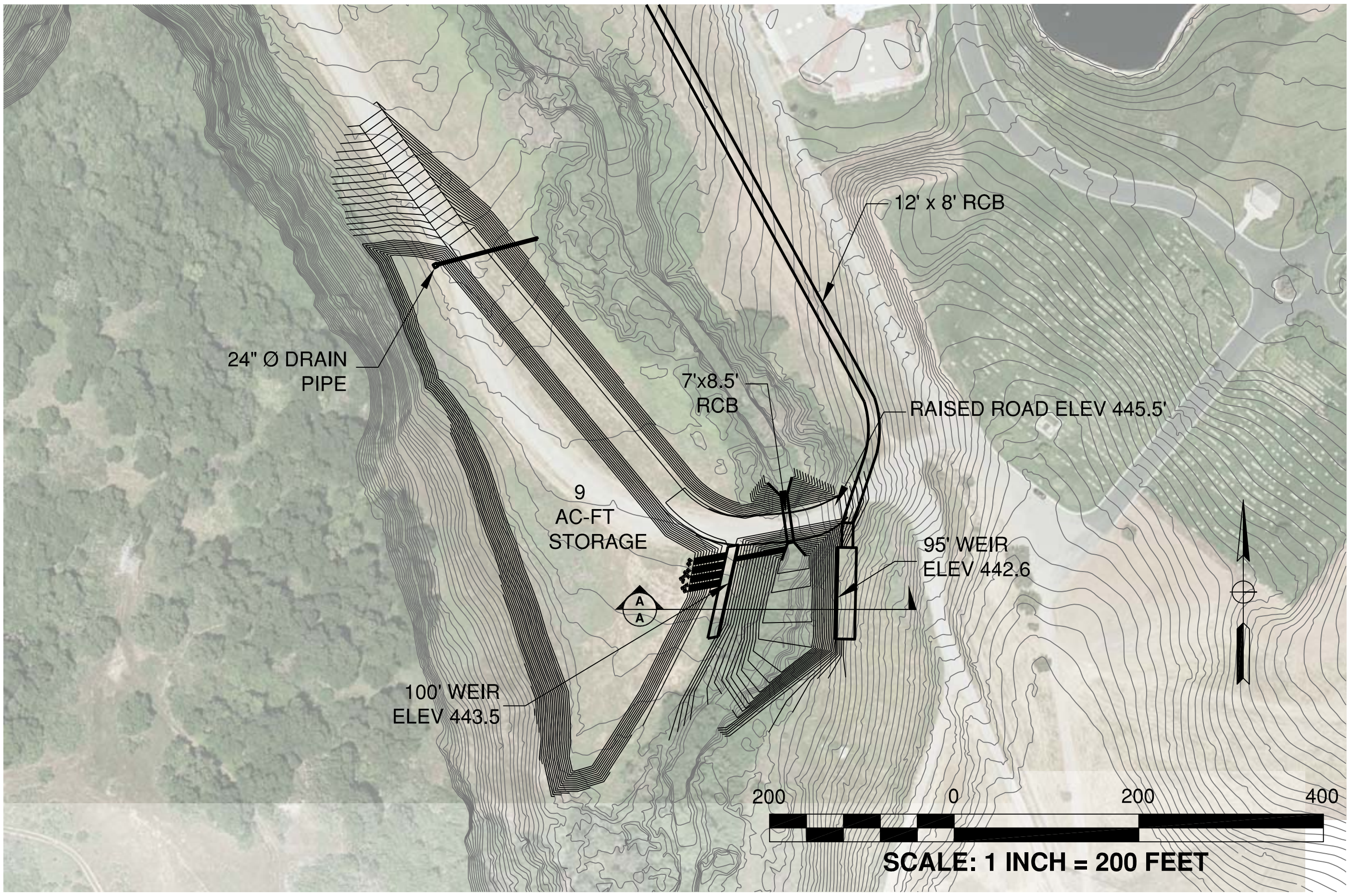
10. U.S. Dept. of Transportation, Federal Highway Administration, Stream Stability at Highway Structures (HEC20), 3rd Ed., 2001.

APPENDICES

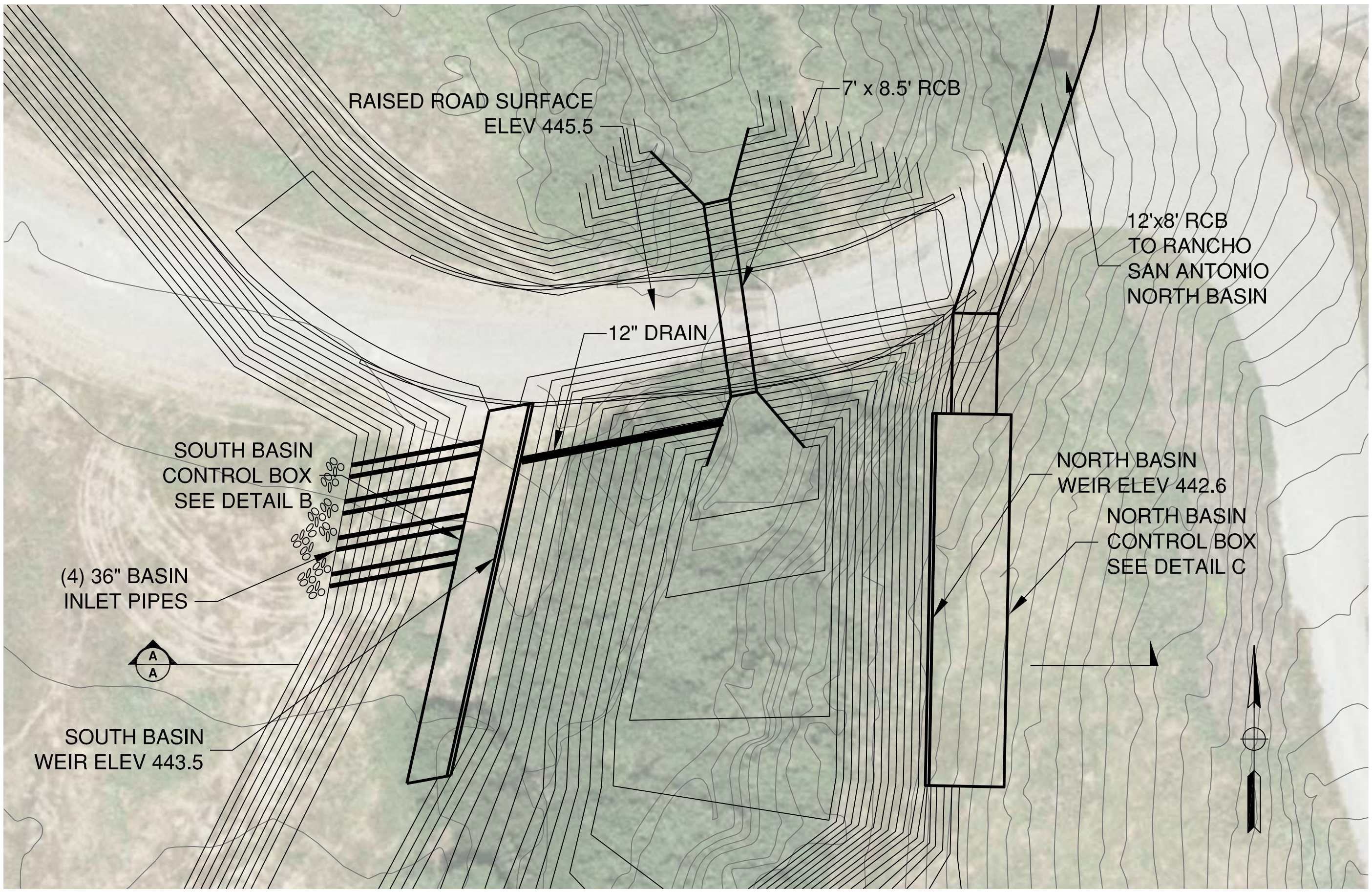
APPENDIX A
RANCHO SAN ANTONIO



RANCHO SAN ANTONIO NORTH

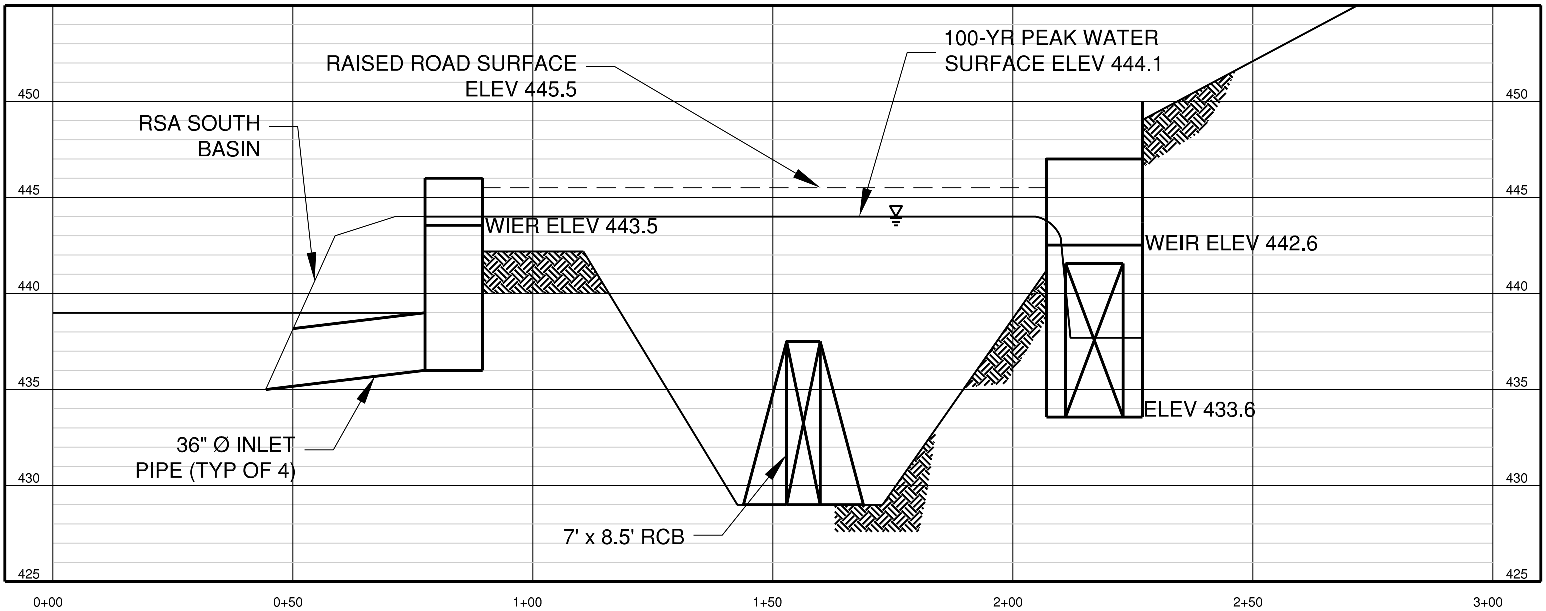


RANCHO SAN ANTONIO SOUTH



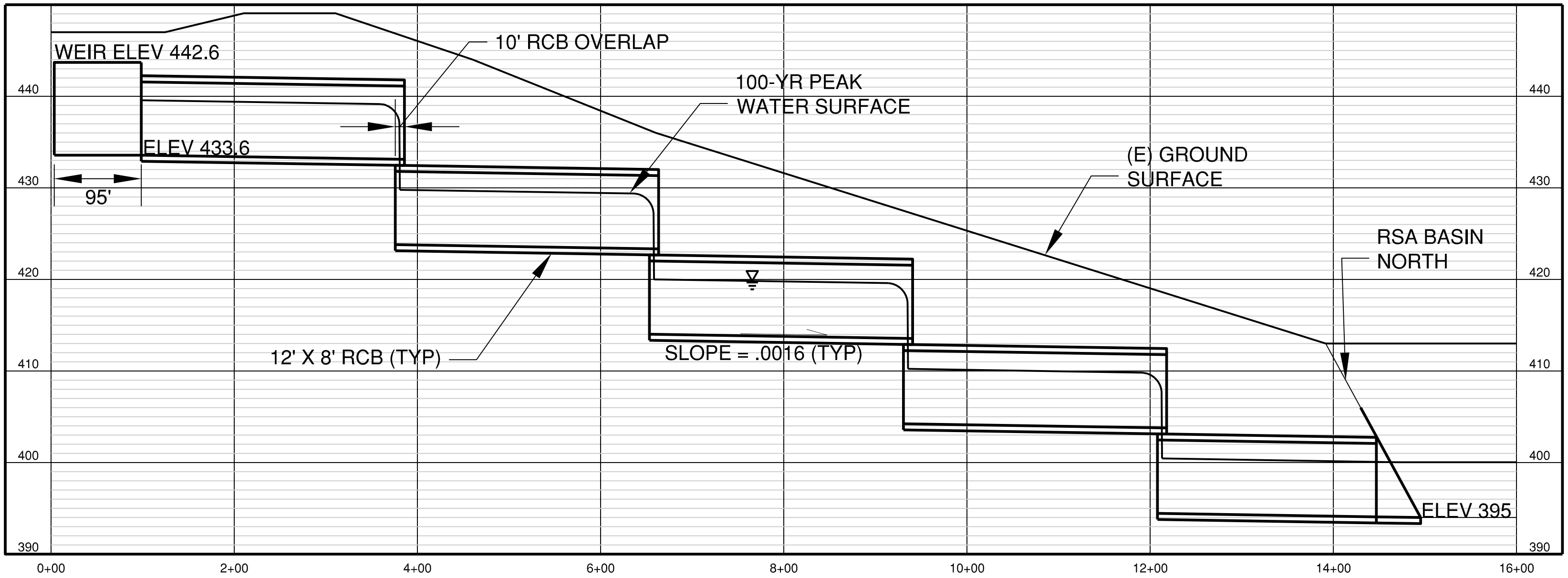
RANCHO SAN ANTONIO DIVERSION

HOR. SCALE: 1" = 50'



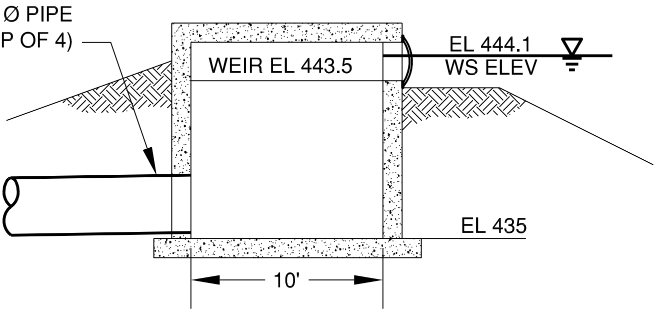
SECTION A

HOR. SCALE: 1" = 40'
VER. SCALE: 1" = 10'



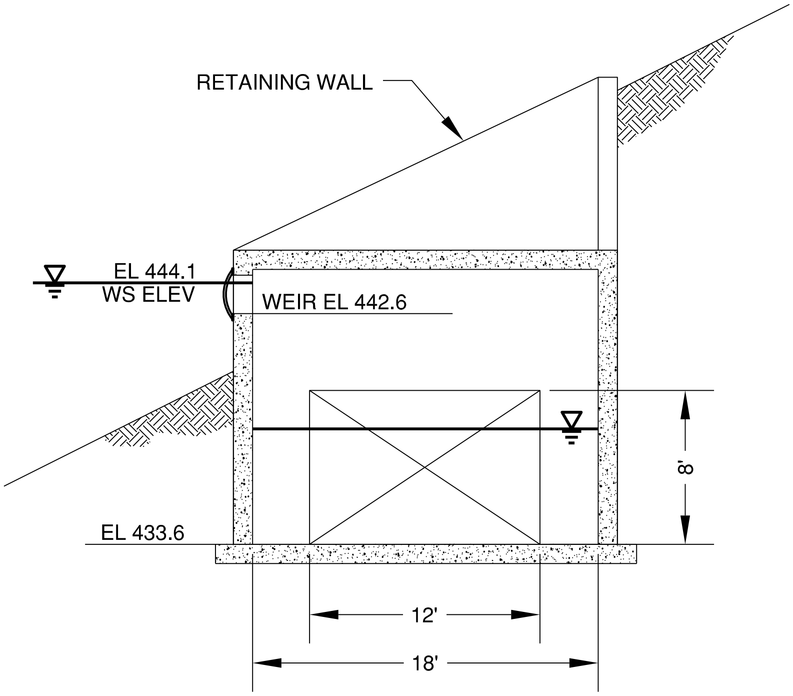
12' x 8' RCB PROFILE AT RANCHO SAN ANTONIO PARK

HOR. SCALE: 1" = 200'
VER. SCALE: 1" = 20'



DETAIL B - SOUTH BASIN CONTROL BOX

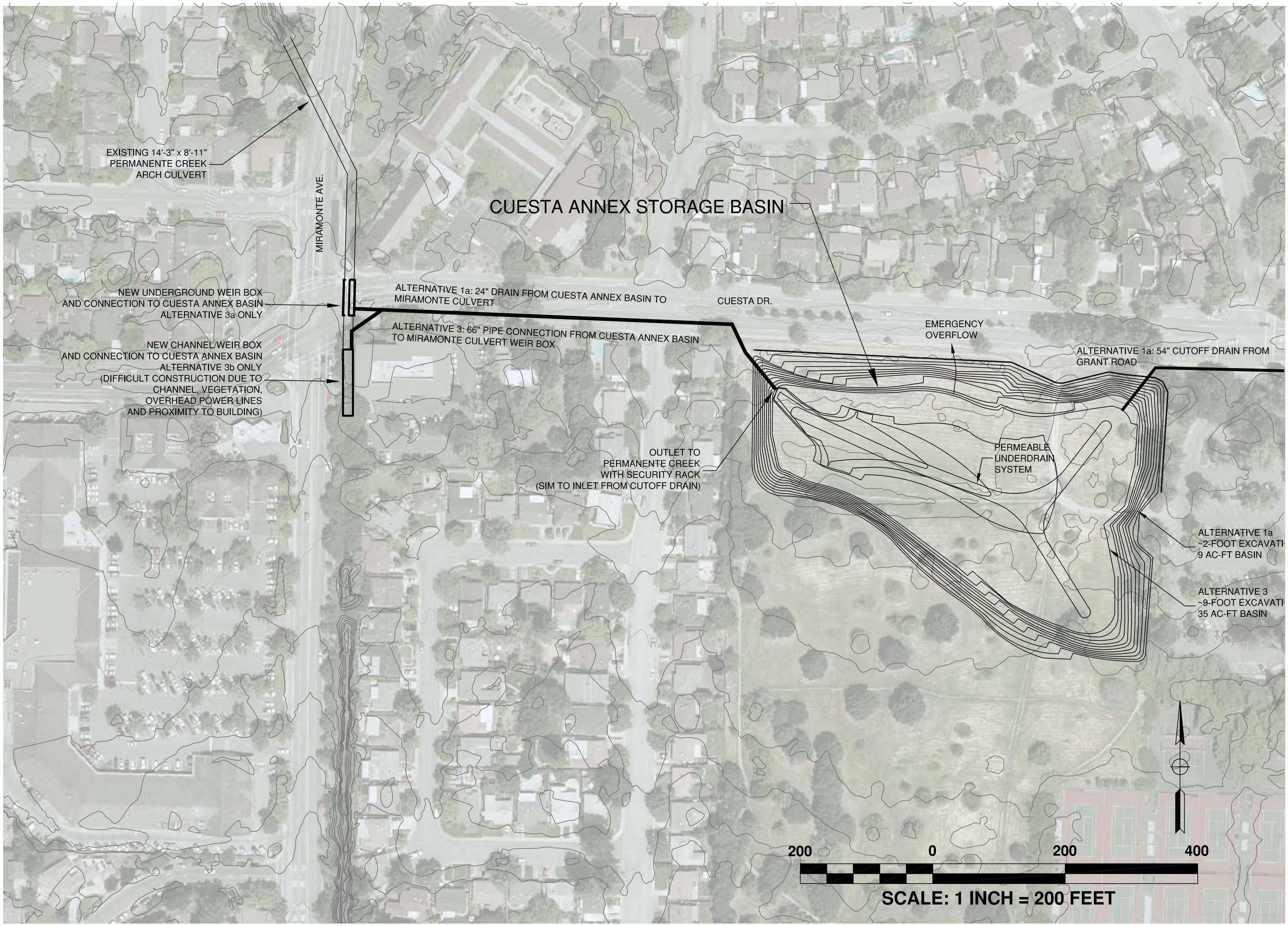
NTS



DETAIL C - NORTH BASIN CONTROL BOX

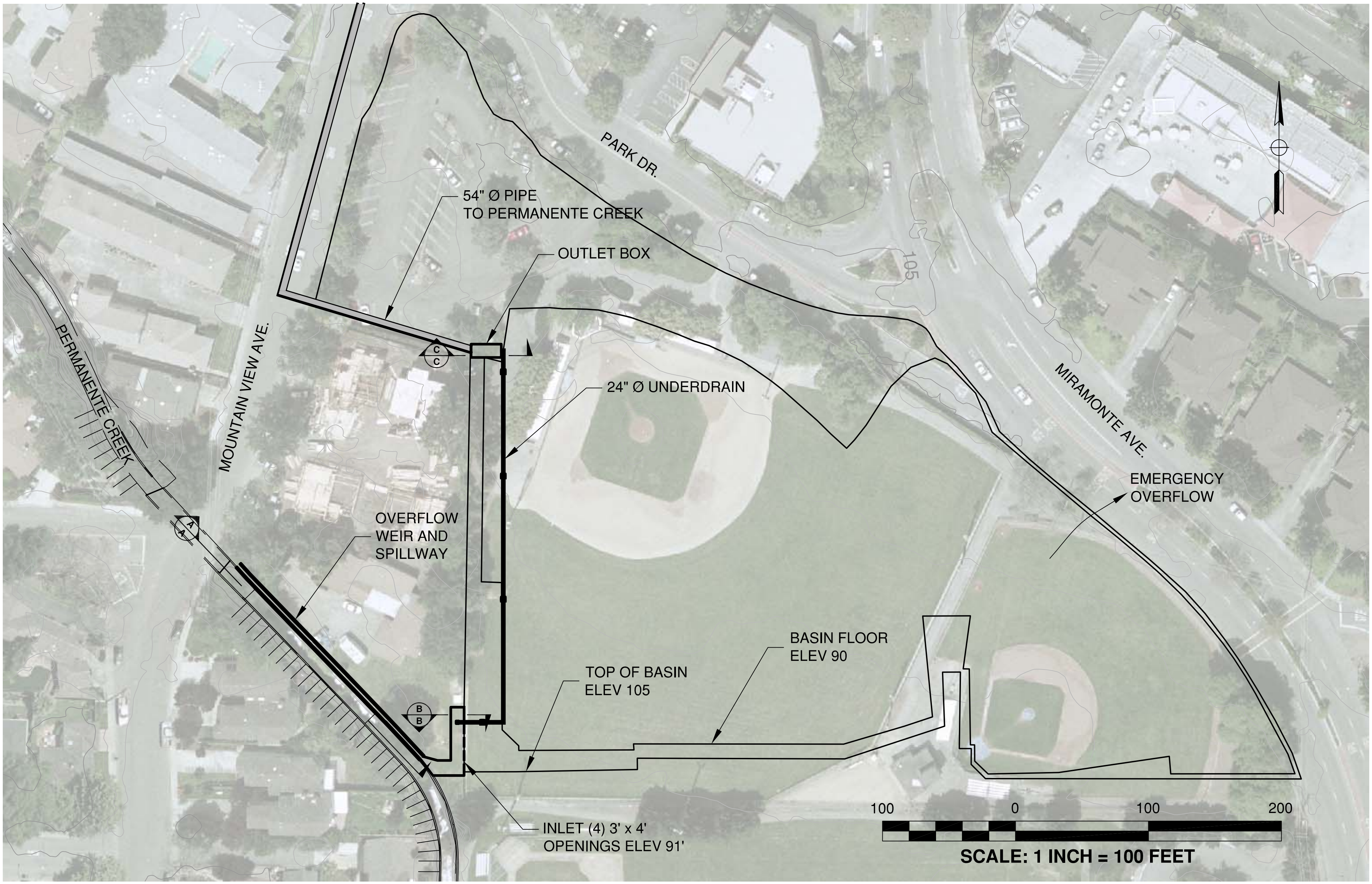
NTS

APPENDIX B
CUESTA PARK ANNEX



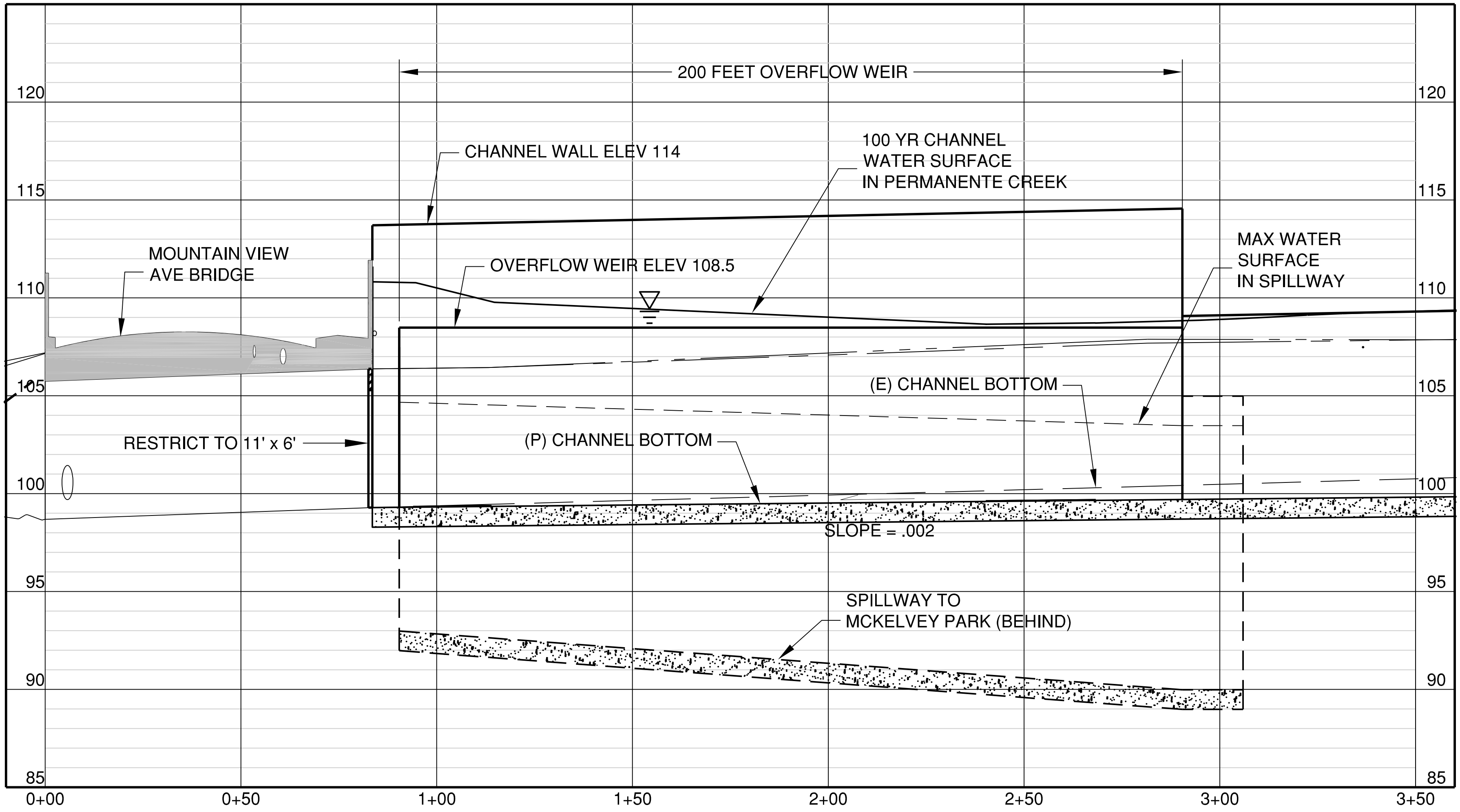
CUESTA PARK ANNEX BASIN

APPENDIX C
MCKELVEY PARK



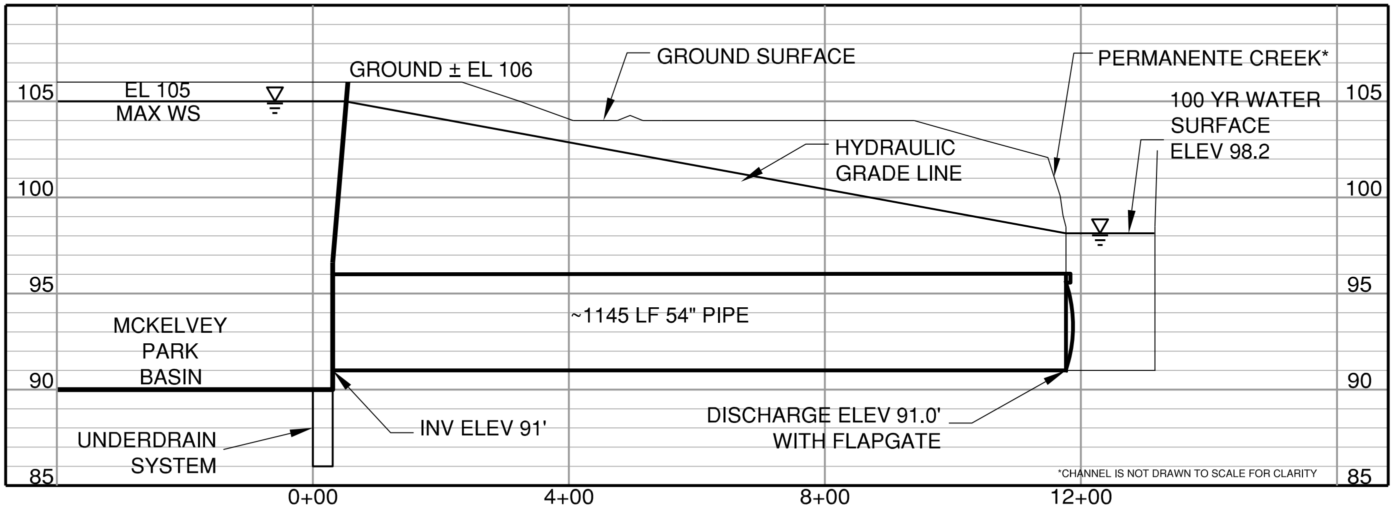
MCKELVEY PARK

SCALE: 1' = 100'



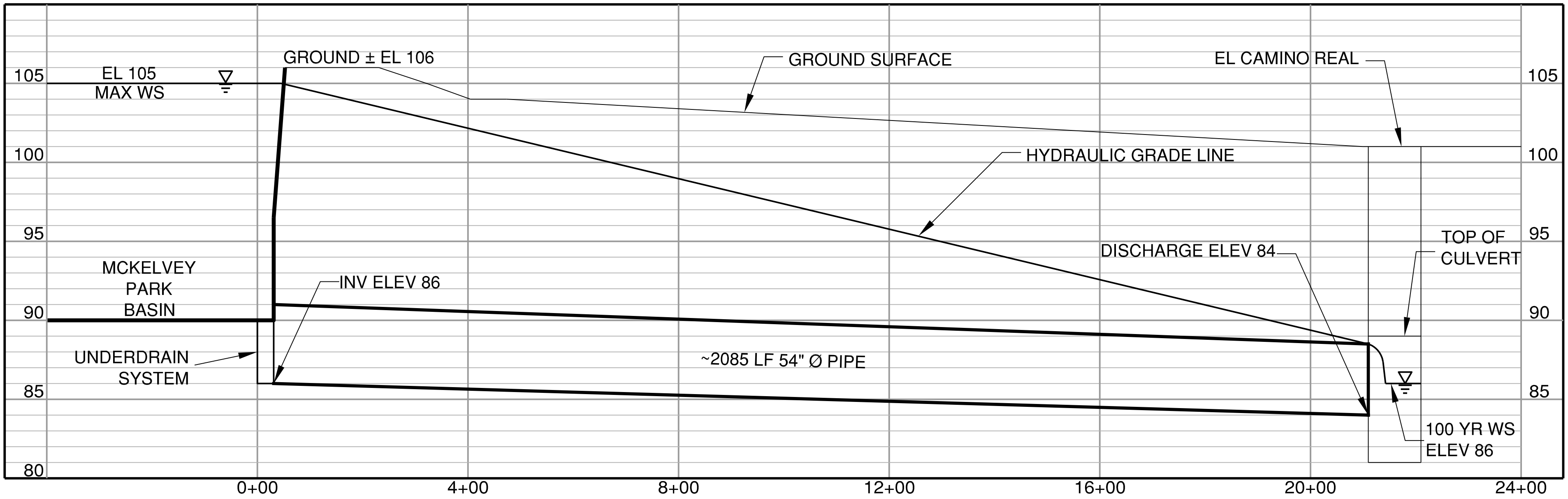
SECTION A - MCKELVEY WEIR PROFILE

HOR. SCALE: 1" = 40'
VER. SCALE: 1" = 8'



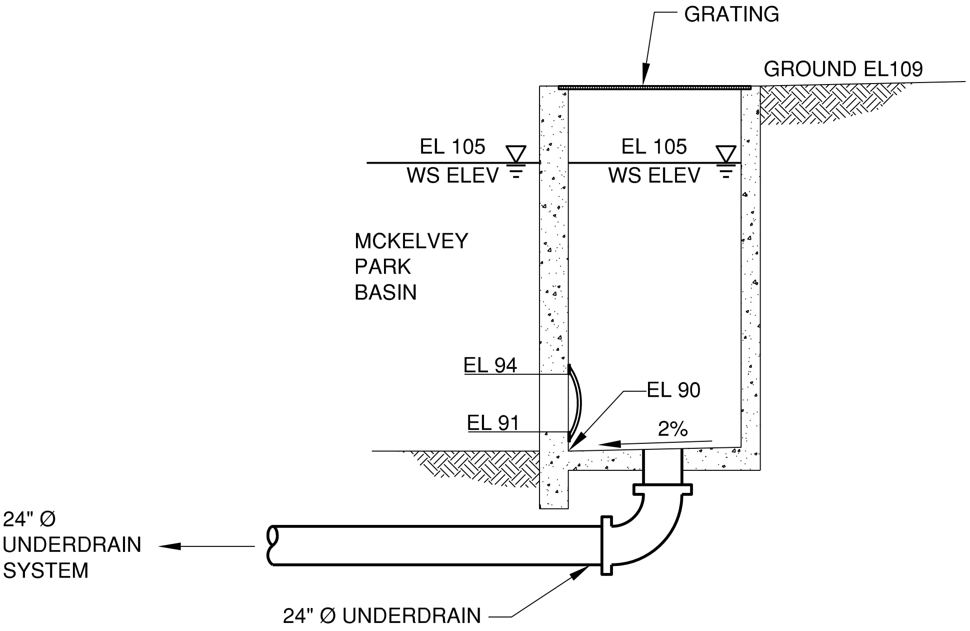
OPTION A: PARK DRIVE OUTFALL

HOR. SCALE: 1" = 300'
VER. SCALE: 1" = 10'



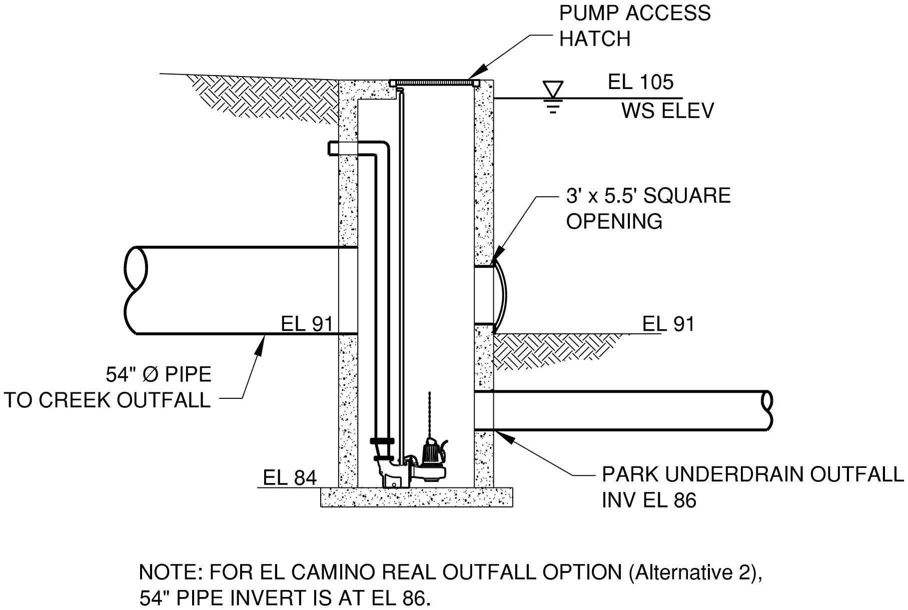
OPTION B: EL CAMINO REAL OUTFALL

HOR. SCALE: 1" = 300'
VER. SCALE: 1" = 10'



SECTION B - MCKELVEY INLET

NTS



NOTE: FOR EL CAMINO REAL OUTFALL OPTION (Alternative 2),
54" PIPE INVERT IS AT EL 86.

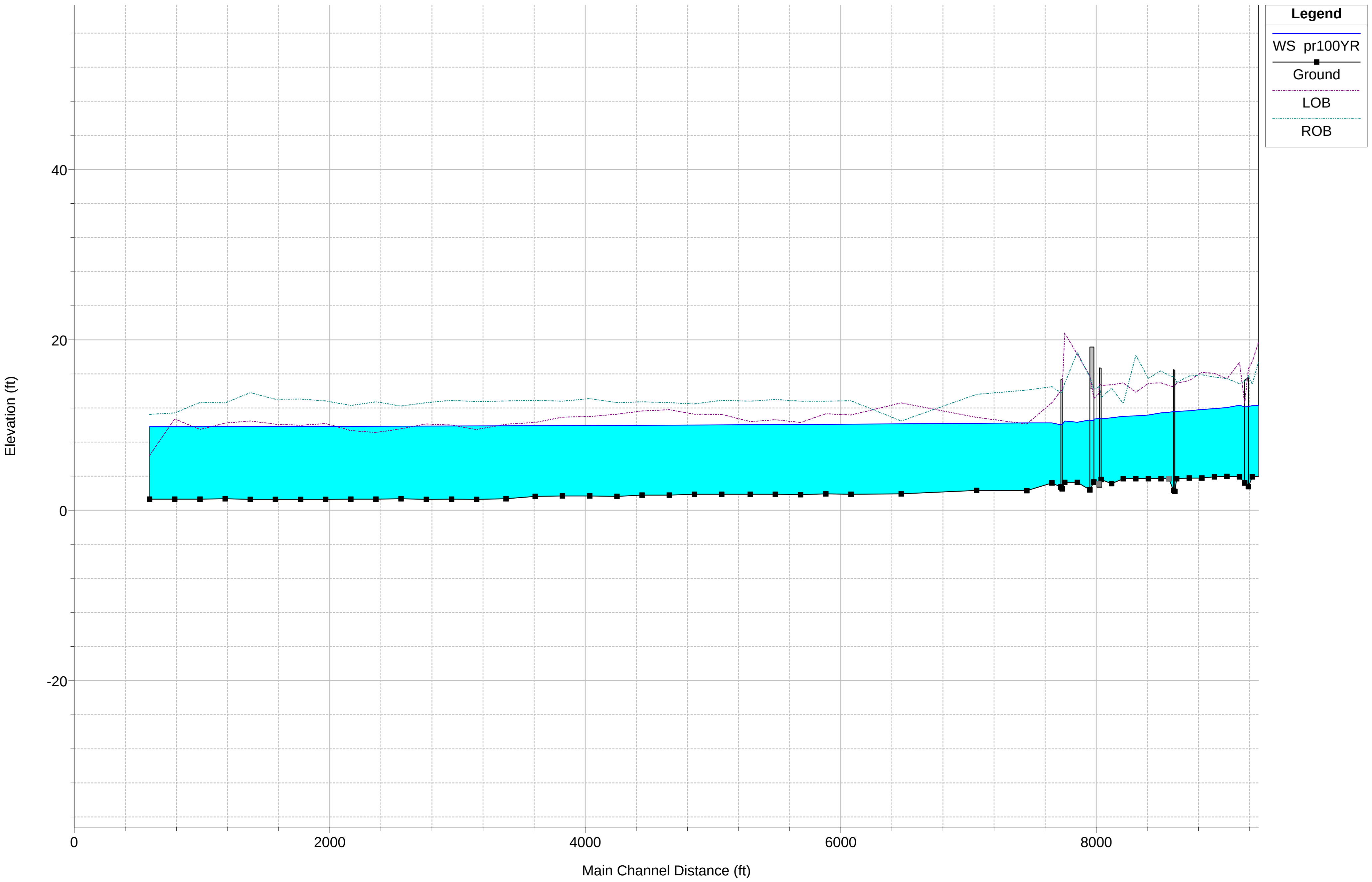
SECTION C - MCKELVEY OUTLET

NTS

APPENDIX D
PERMANENTE CREEK 100 YR WATER SURFACE PROFILES

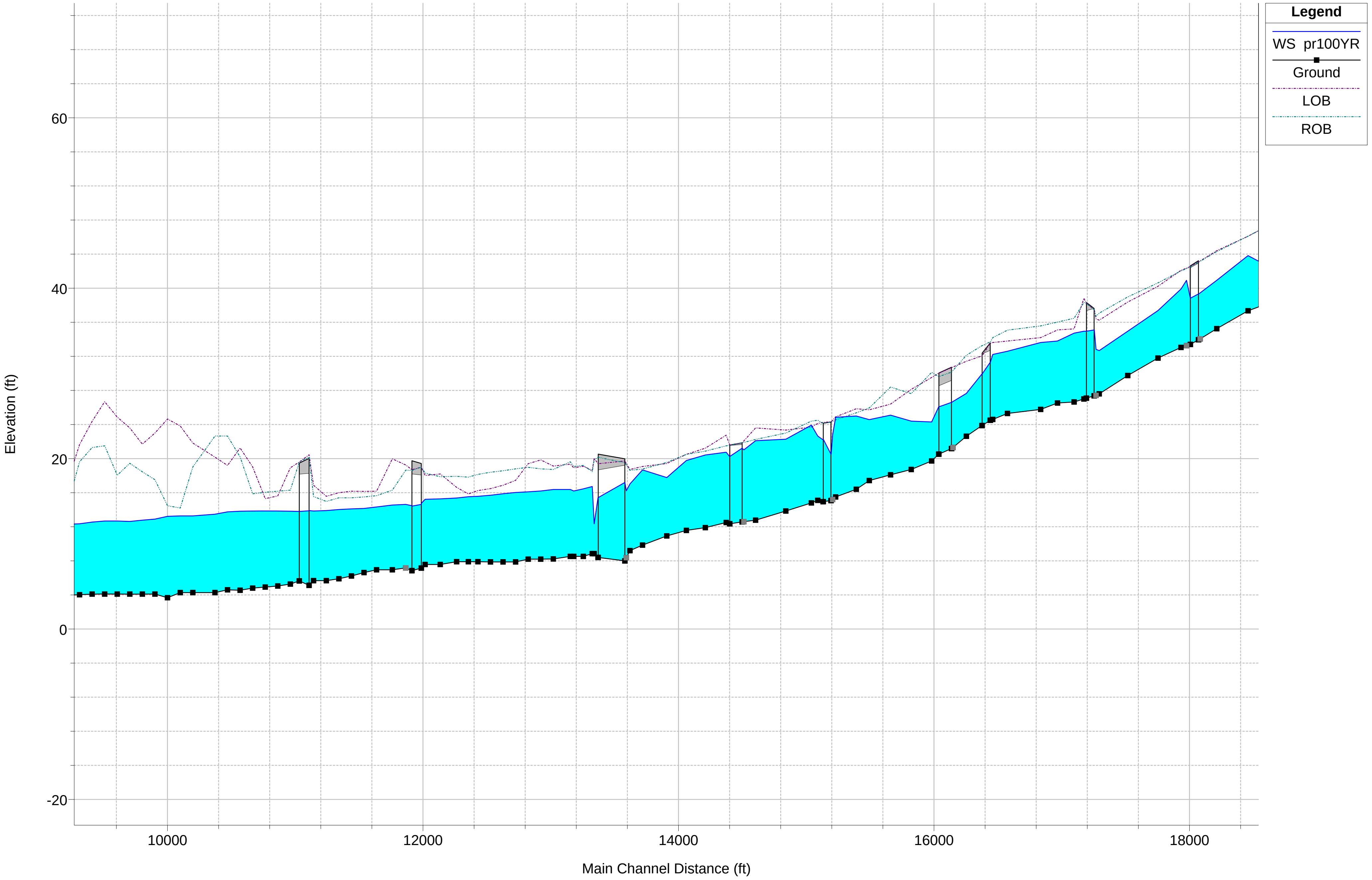
DOWNSTREAM PERMANENTE Plan: 100 YR FINAL

November 2011

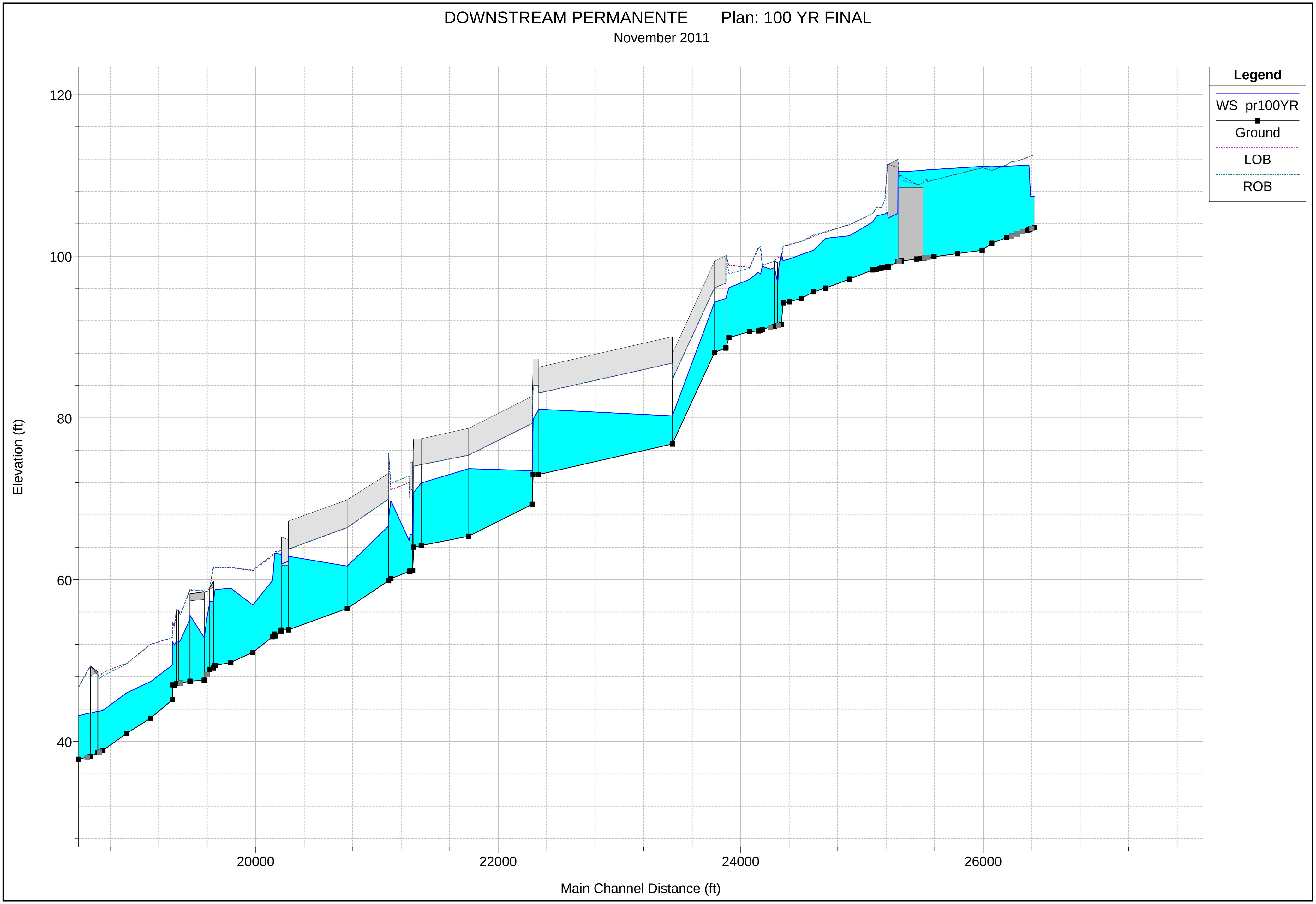


1 in Horiz. = 1000 ft 1 in Vert. = 15 ft

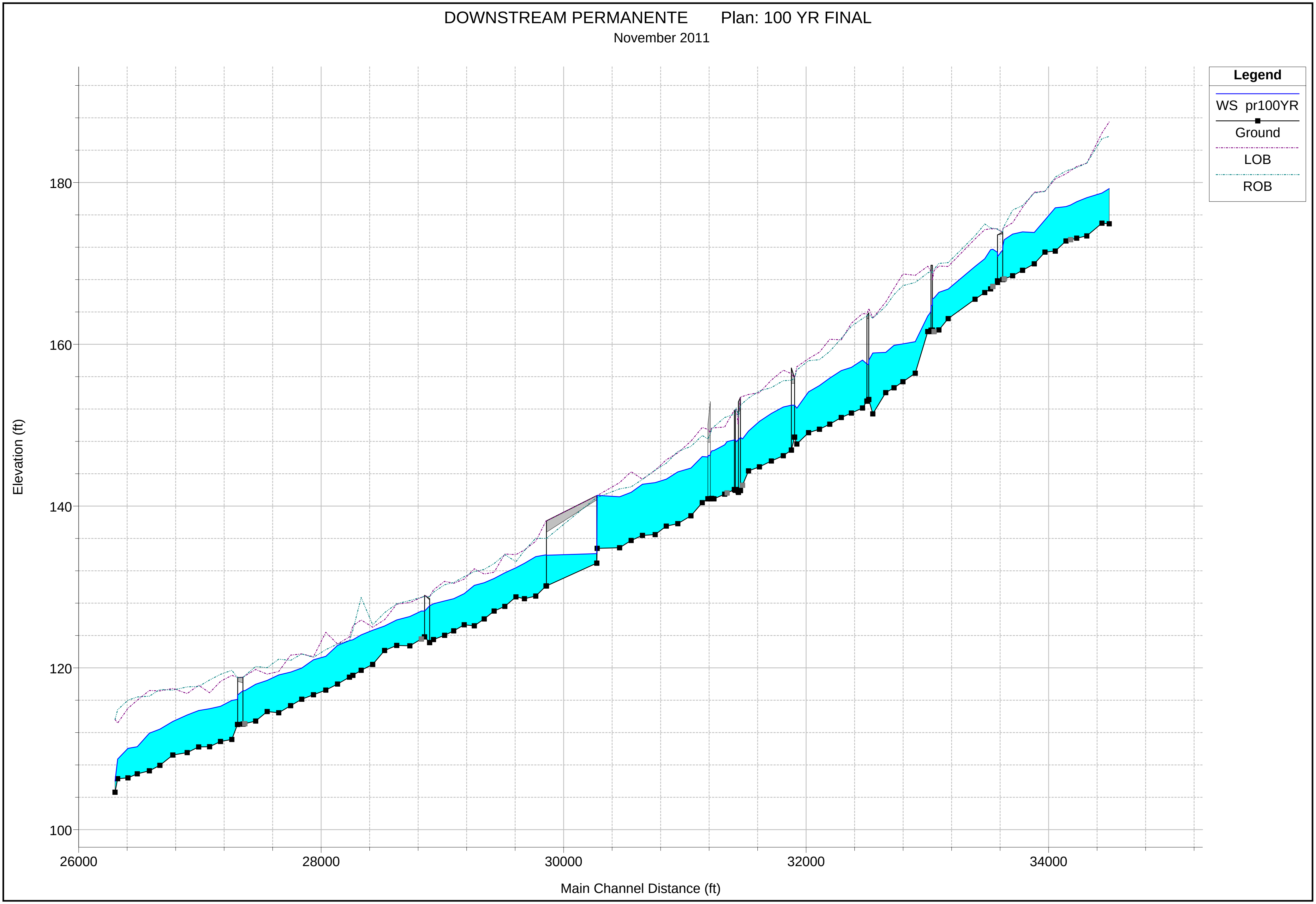
DOWNSTREAM PERMANENTE   Plan: 100 YR FINAL
November 2011



1 in Horiz. = 1000 ft   1 in Vert. = 15 ft



1 in Horiz. = 1000 ft 1 in Vert. = 15 ft

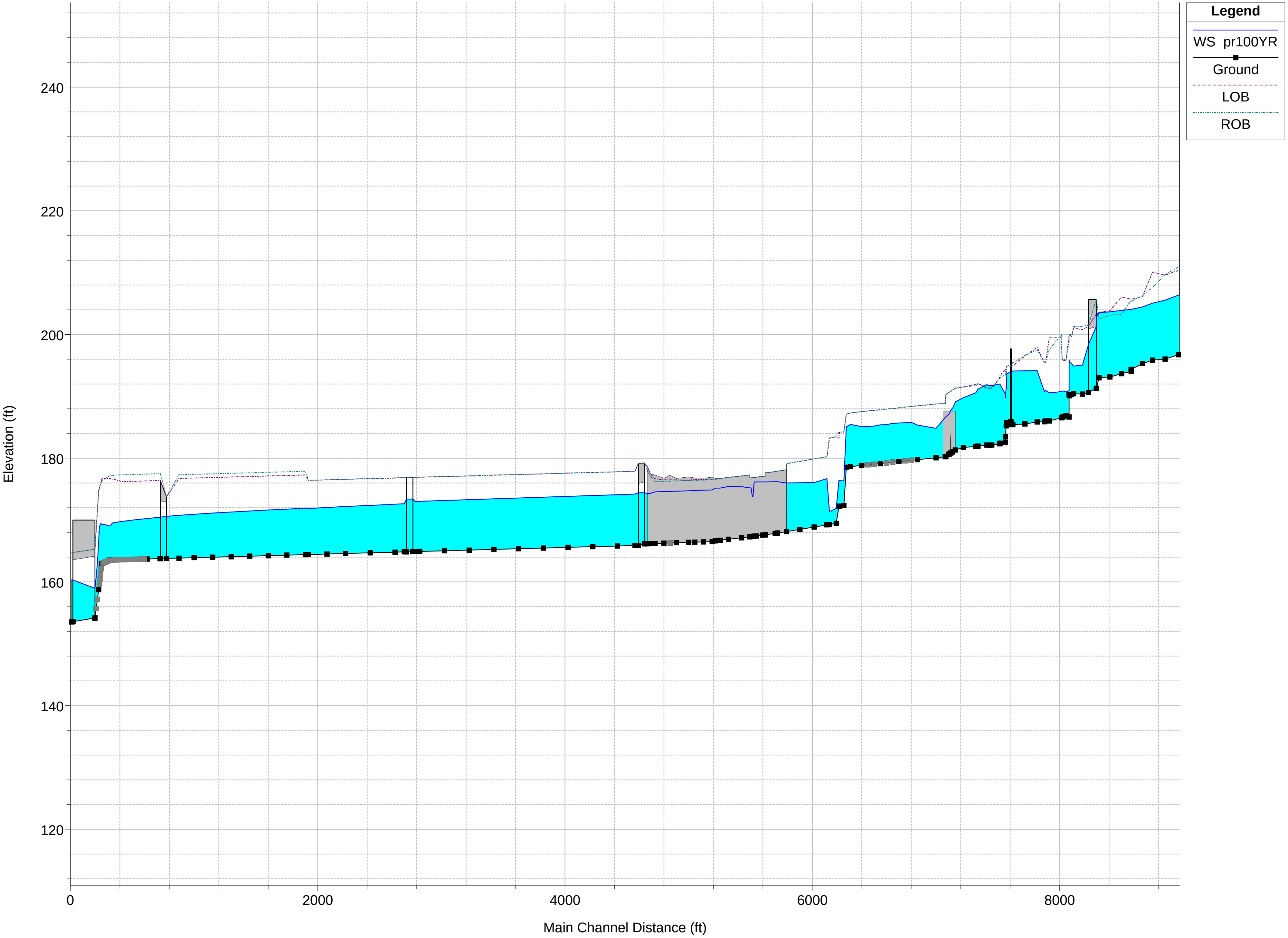

DOWNSTREAM PERMANENTE Plan: 100 YR FINAL

November 2011

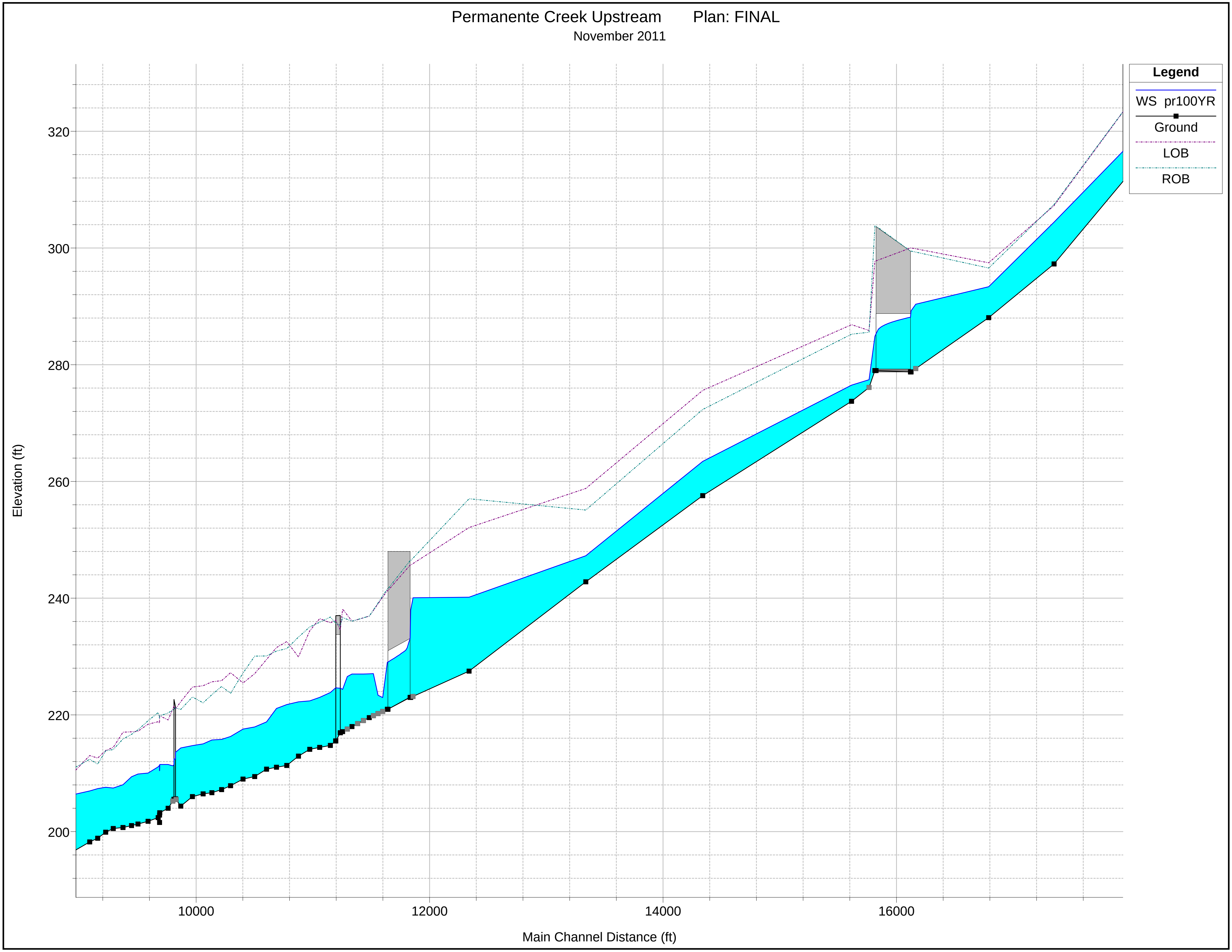
1 in Horiz. = 1000 ft 1 in Vert. = 15 ft

Permanente Creek Upstream Plan: FINAL

November 2011



1 in Horiz. = 1000 ft 1 in Vert. = 20 ft



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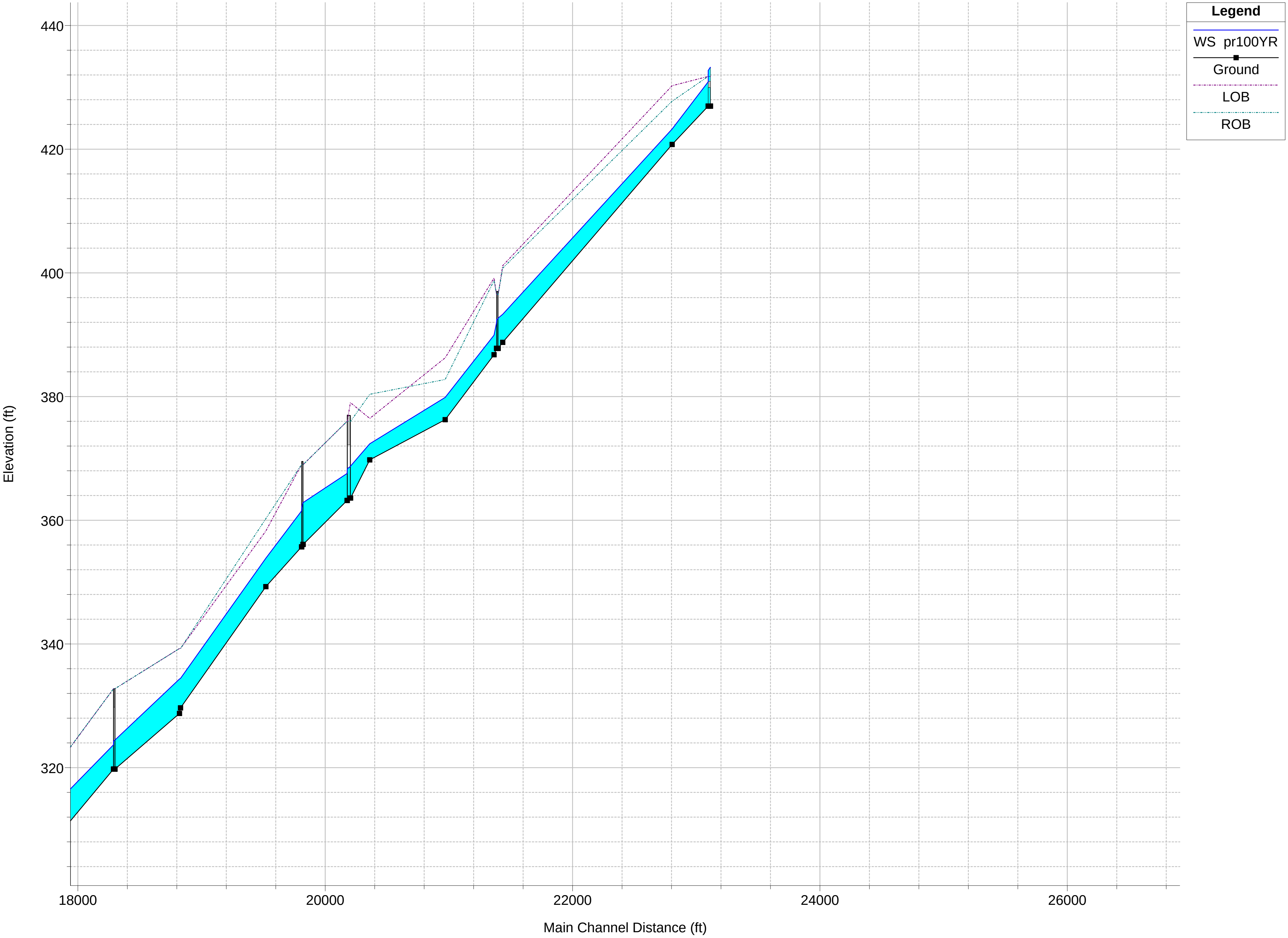
Permanente Creek Upstream Plan: FINAL

November 2011



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